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and

North Atlantic Treaty Organization/Committee on the Challenges of Modern Society Conference on Sewage Treatment Technology

Volume I. Part A. Japanese Papers

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16. ABSTRACT

This publication is one of three volumes that contain Japanese, American, and NATO papers presented at the Tenth United States/Japan Conference on Sewage Treatment Technology and the North Atlantic Treaty Organization/Committee on the Challenges of Modern Society (NATO/CCMS) Conference on Sewage Treatment Technology held in Cincinnati, Ohio, October 1985. The proceedings of the conferences comprise a useful body of knowledge on advances in sewage and wastewater treatment and sludge management technology. Subject matter covered includes municipal wastewater treatment including bioengineering applications and anaerobic treatment; sludge treatment and disposal including combustion, oxidation, composting, and toxics removal; odor control; small flow sewerage system; sludge conversion to oil, etc. Volume I, Part A, contains the Japanese Papers; Volume I, Part B, contains the American Papers; and Volume II contains the NATO Papers.

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PROCEEDINGS

TENTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

OCTOBER 17-18, 1985

AND

NORTH ATLANTIC TREATY ORGANIZATION/COMMITTEE ON THE CHALLENGES OF MODERN SOCIETY (NATO/CCMS) CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

OCTOBER 15-16, 1985

CINCINNATI, OHIO

VOLUME I.

PART A. JAPANESE PAPERS

U.S. ENVIRONMENTAL PROTECTION AGENCY
OFFICE OF RESEARCH AND DEVELOPMENT
CINCINNATI, OHIO 45268

NOTICE

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FOREWORD

The maintenance of clean water supplies and the management of municipal and industrial wastes are vital elements in the protection of the environment.

The participants in the Japan-United States-North Atlantic Treaty Organization/Committee on the Challenges of Modern Society (NATO/CCMS) Conferences on Sewage Treatment Technology completed their conferences in Cincinnati, Ohio, in October 1985. Scientists and engineers of the participating countries were given the opportunity to study and compare the latest practices and developments in Canada, Italy, Japan, The Netherlands, Norway, the United Kingdom and the United States. The proceedings of the conferences comprise a useful body of knowledge on sewage treatment which will be available not only to Japan and the NATO/CCMS countries but also to all nations of the world who desire it.

Lee M. Thomas
Administrator

Washington, D.C.

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VIEW OF CAPTOR WASTEWATER TREATMENT PROCESS
TEST AND EVALUATION FACILITY, CINCINNATI, OHIO

VISIT TO THE MULTIPLE DIGESTION PROJECT,
TEST AND EVALUATION FACILITY, CINCINNATI, OHIO

JOINT COMMUNIQUE

TENTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

Cincinnati, Ohio
October 18, 1985

1. The Tenth United States/Japan Conference on Sewage Treatment Technology was held in Cincinnati, Ohio, from October 17 to 18, 1985.

2. The Japanese delegation headed by Dr. Takeshi Kubo, Counselor, Japan Sewage Works Agency, was composed of two representatives from the Ministry of Construction, three representatives from the Japan Sewage Works Agency and one each from the local governments of Tokyo, Yokohama and Fukuoka.

3. Mr. John J. Convery, Director, Wastewater Research Division, Water Engineering Research Laboratory, U.S. Environmental Protection Agency, was head of the U.S. delegation, which consisted of seven representatives of the federal government, five academia representatives and three representatives from consulting engineering firms and scientific laboratories.

4. The chairmanship of the Conference was shared by Mr. John J. Convery and Dr. Takeshi Kubo.

5. During the Conference, papers relating to the joint research projects on sludge treatment and disposal, including combustion, oxidation and composting, were presented by both sides. Data and findings on the joint research projects were mutually useful and provided increasing insights into the nature of the problems and potential solutions for each country. A decision was made to expand the scope of the joint research projects to include anaerobic treatment of wastewater.

6. Principal topics of the Conference were bioengineering applications in wastewater treatment as well as sludge management and disposal, aeration practice, wastewater reuse, odor control, small flow sewerage system, nutrient control and innovative biological treatment processes.

The discussions which followed the presentations were also useful to both countries.

7. Field visits in Lawrence, Marlborough and Hartford, Connecticut; Chicago, Illinois; Madison and Milwaukee, Wisconsin; and Sacramento, California; are planned to inspect wastewater treatment facilities in these areas.

8. Recent engineer exchanges between the two countries included a two-week visit in 1985 to Japan by Mr. James F. Kreissl, Wastewater Research Division, Water Engineering Research Laboratory, U.S. Environmental Protection Agency, and a fourteen-month visit to the United States by Dr. Kazuhiro Tanaka, Japan Sewage Works Agency, in 1984 to 1985. Mr. Takashi Kimata of the Japan Sewage Works Agency is now staying at the above U.S. EPA Cincinnati Research Laboratory. Both parties agreed to continue the engineer exchange program.

9. It was proposed by the Japanese side that the Eleventh Conference be held in Tokyo, Japan, about September 1987, and the future Conferences in the United States would be held in Cincinnati, Ohio and also in Washington, D.C. as were the past Conferences.

10. A proceedings of the Conference will be printed in English and Japanese.

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Tenth United States/Japan Conference
on Sewage Treatment Technology

APPLICATION OF BIOTECHNOLOGY
TO
WASTEWATER TREATMENT

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APPLICATION OF BIOTECHNOLOGY TO WASTEWATER TREATMENT

Itaru Nakamoto
Head
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PREFACE

In Japan, secondary treatment of municipal wastewater became real for the first time using standard-rate trickling filter in Tokyo in 1922. However, the activated sludge process became the leading one among wastewater treatment processes since the conventional process was introduced in Nagoya-city in 1930. The activated sludge process requires comparatively high skills and attentive control of the sludge to perform good treatment, while it offers an advantage creating the better-quality effluent than any other secondary processes.

Despite the long history of success, improvement of the activated sludge process has been strongly desired since it requires an immense land or enormous energy for treatment and disposal of excess sludge. In addition, the effluent is expected to play a positive role in the prevention of eutrophication of lakes and oceans, thereby increasing the need for development of the technology to improve and stabilize the removal rate of nitrogen and phosphorus which cause eutrophication. Further, various needs for technology development are imposed on us, such as the technology to degradade and stabilize toxic substances when subtley contained in the influent, decoloring technology which widens the scope of reuse of the effluent, and energy saving wastewater treatment technology which is the eternal need of Japan poor in its energy resources.

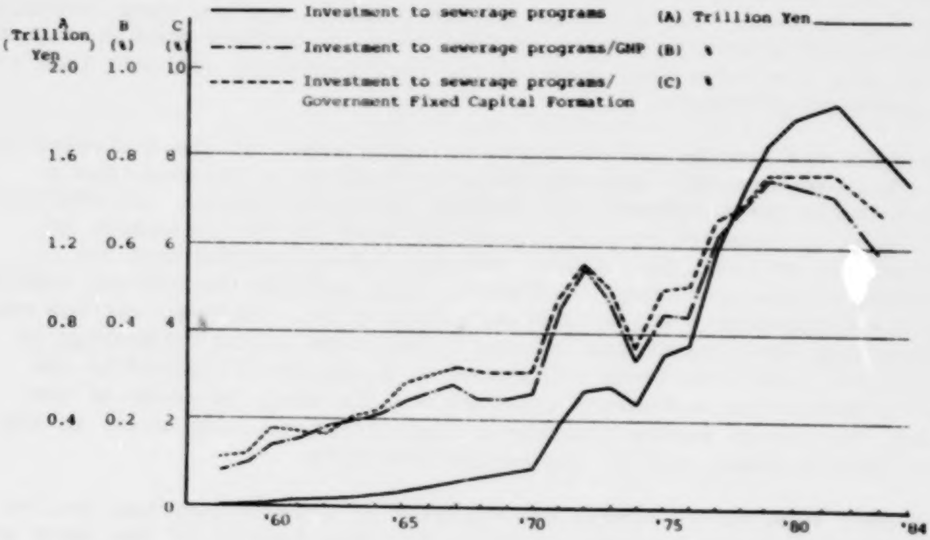
To meet such social needs, the Ministry of Construction has decided to carry out the five-year research project for "Development of new wastewater treatment systems employing biotechnology", starting from the fiscal year of 1985.

This project is called "Biofocus W.T.". Described in this paper are the status quo of the Sewerage and Sewage Purification Programs in Japan which helped to set out the biotechnology project and to identify the needs for wastewater treatment technology (Chapter 1), and the outline of the Biofocus W.T. (Chapter 2).

1. STATUS QUO OF SEWERAGE AND SEWAGE PURIFICATION PROGRAMS IN JAPAN AND NEEDS FOR WASTEWATER TREATMENT TECHNOLOGY

1.1 History and Status Quo of Sewerage and Sewage Purification Programs

The Sewerage and Sewage Purification Programs as public works became real with enactment of the Sewerage Law in 1958. The first Five-Year Plan of Sewerage Construction was started in 1963. Since then, several Five-Year Plans have been carried out with revisions until the current 5th Five-Year Plans. During such period, the scale of investment to the sewerage programs went through rapid progress. It had once grawn equivalent to the defense budget. However, it has been at a standstill these years reflecting the severe financial situations (Fig. 1).



Note: GNP and government fixed capital formation of each year before 1964 are based on the old SNA.

Fig. 1 Trends of the investment to sewerage programs

The number of municipalities having started Municipal Sewerage Program is 836 at the end of FY 1984. This number holds a quarter of the total 3,256 Japanese municipal bodies. Four-hundred and eighty-three (483) municipalities out of such 836 are currently carrying out treatment of their municipal wastewater. Regional sewerage program has been carried out at 81 places in the 41 prefectural divisions, and 43 wastewater treatment plants are now in operation. Moreover, the Special Environment Protection Municipal Sewerage Program for agricultural/mountain/fishing villages and national parks have been carried out at 94 places, and the Special Public Sewerage Program for the areas containing industrial wastewater over two third (2/3) of the total quantity have been practiced at 8 places (Fig. 2).

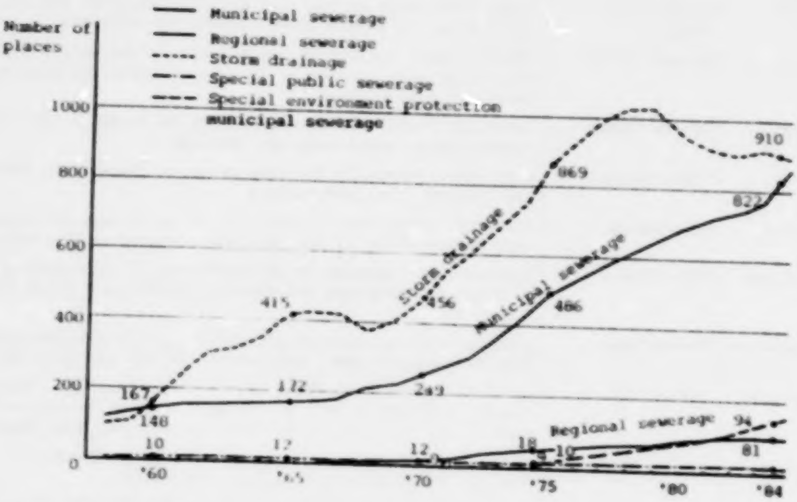


Fig. 2 Trend of the number of cities carrying out sewerage programs

In spite of such increase in the investment to the Sewerage and Sewage Purification Programs and in the number of places carrying out sewage works, improvement has been slow in the spread of sewerage systems. The fiscal year of 1984 has proved that the number of inhibitants in the sewered areas holds no more than 34% of the total Japanese population. Such percentage held by designated cities with population over one million is as high as 80%, while general cities show such percentage of only 20%. This means that there is a big disparity in the service by sewerage systems among areas. It is urgently demanded to spread sewerage systems to comparatively less-inhibitied local cities thereby decreasing the disparity among areas.

The Ministry of Construction has announced several measures in these years, to let as many people as possible understand the benefits of sewerage systems and to smoothly promote the sewerage programs. Table 1 presents the list of cities to which the model project of sewage works called Aquatopia (from Aqua Utopia) are applied and the purposes of the project. In these works, it is tried to retrieve "rest by the water". In the Aquatopic cities, the sewerage programs are to be promoted with priority. Besides such works, the other model projects such as Appeal Sewerage (Table 2) and Idea sewerage (Table 3) have been practiced, thereby trying to make citizens recognize the benefit of sewage works.

Table 1 List of aquatopia projects

Prefecture	City/Town	Major water body	Major purpose
Hokkaido	Ikeda	Kiyomi Nisen River	Clean water ways in the town and improve water quality of the Kiyomi Nisen River to create resort places for citizens.
Aomori	Hirosaki	Dobuchi River	Improve water quality of the streams in the city and stock the water ways with carp so as to give citizens the opportunities to familiarize with water.
Yamagata	Tendo	Kuratsu River	Clean all water ways in the city so as to offer the citizens the resort places where carp are stocked.
Ibaraki	Itako	Itako Cannals	Clean all cannals in the town so as to improve the environment of the town of "Iris and Cannals.".
Gunma	Haruna	Lake Haruna	Clean all water ways in the town so as to maintain water quality of Lake Haruna which is the precious property of the citizens.
Gifu	Takayama	Miya River	Improve water quality of all water ways in the city so as to maintain the precious environment of the town which is called "small Kyoto".
Shizuoka	Hamamatsu	Hamana Lake	Improve water quality of all water ways in the city so as to offer the citizens the opportunities to familiarize with water.
Osaka	Toyonaka	Intown Rivers	Restore the "home of fireflies" at the artificial water way in the Tokura district as well as to improve water quality of all water ways in the city so as to offer the citizens the resort places.
Shimane	Matsue	Ohashi River	Improve water quality of all water ways in the city so as to restore the Matsue-city as "city of water".
Okayama	Kurashiki	Kurashiki River	Improve water quality of water ways in the city so as to create the environment matching the white walls (symbol of the city) and to offer the citizens the opportunities to familiarized with water.
Yamaguchi	Hagi	Niihori River	Improve water quality of all water ways in the city so as to restore the precious environment of Hagi, a historical castle town.
Fukuoka	Yanagawa	Yanagawa Cannals	Improve all water ways in the city so as to restore the environment of the cannal district which is remembered with connection to Hakushu (historical Japanese poet), the property of the citizens.
Kumamoto	Hitoyoshi	Yamada River	Maintain water quality of Kuma River (famous for rifting) as well as to improve water quality of all water ways in the city so as to offer the citizens the opportunities to familiarized with water.

Table 2 List of appeal sewerage projects

Prefecture	City/Town	Project name	Purposes
Hokkaido	Otaru	Otaru Cannal Clean Project	The Otaru cannal district along with the stone warehouses, the remainder of Hokkaido pioneering era, is a resort spot for the citizens. The cannal whose water quality has been deteriorated by domestic wastewater shall be cleaned via urgent preparation of sewage works so as to protect the historical legacy.
Hokkaido	Lake Akan	"Marimo" (Aegagropila Sauteri) Protection Sewerage System	Water quality of the mysterious Lake Akan where "marimo" (Aegagropila Sauteri) is found has been deteriorated every year due to urbanization of the peripheral areas. Miscellaneous wastewater from the hotel district around Lake Akan, which is now discharged into the lake, shall be taken into sewage so as to protect the precious natural product "marimo" from now on as well as in future.
Saitama	Honjo	Friendly to Weaver Fish (Tittlebat) Sewerage System	Moto-oyama River which is completely polluted by urbanization need be cleaned by preparation of sewage so as to create a friendly environment for weber fish that used to live therein and to complete the "Honjo Riverside Combination Park" which links historical remains and sights along Moto-oyama River.
Toyama	Inami	Snowless Sewerage	Inami-machi has snow over 1 m depth every winter, which affects traffic and civil life in the town. It is promoted to make the city tough against snowing by providing the snow melting function (Removing snow from the town) to the Yamami-city sewage system.
Saitama	Kawagoe	Firefly Fostering Shingashi River Project	The Kawagoe-city municipal sewerage shall be prepared so as to clean water of Shingashi River. The natural environment, in which fireflies show up as dusk gathers, shall be restored together with the civil movement for "Firefly Fostering Shingashi River".
Osaka	Toyonaka	Preparation of the Firefly Dancing Sewerage System	Urbanization has been promoted thereby spoiling the natural environment. The natural environment shall be restored so that fireflies dances in the Tokura district (which used to be famous for fireflies) and the peripheral districts thereof and to create a cozy city space: Artificial breeding of fireflies is available now.
Kyoto	Kyoto	Yodo Castle Sight Protection Sewerage System	The site of Yodo castle is now popular as Yodo castle park. Mixing of miscellaneous wastewater from the peripheral districts into the moat around the castle and water ways in the peripheral districts has deteriorated the water quality. Water environment shall be made suitable for the historical remainings by promptly preparing sewerage in the peripheral districts for improvement of the water quality.
Shiga	Ohmi-hachiman	The Hachiman Cannal Revival Sewerage System	Hachiman cannal established in the era of Momoyama (1585) has been polluted by accumulated sludge causing water quality deterioration. Dredging and plantation for scenic purpose have been practiced in the course of river improvement works and city greening works (by the National Land Agency) with less effect for water quality maintenance. Therefore, the municipal sewerage shall promptly start its services so as to maintain water quality of the Hachiman cannal.
Osaka	Sakai	Revival of Rikyu's Intrenchment	Sakai-city had flourished in the era of civil wars. One of the reasons for prosperity is that the town surrounded itself by intrenclment in fear of invaders and observed strict nutrality in politics and military affairs. Sewage works shall be prompted in hilly districts so as to revive such intrenchment, which is noted with connection to Rikyu (historical Japanese poet).
Okayama	Yamate	Agricultural Villages Waste-water Reclamation Project	Yamate-mura is hardly benefitted by water resources having small quantity of rain through the year and no trunk rivers. Accordingly, treated wastewater shall be utilized for the secondary water ways as well as for agricultural waters.
Ehime	Imabari	Rivers Revival Project by Short Cut - Kinboshi River Polluted by Dyeing	Imabari-city, whicih is the No. 1 in towel production, has been suffered from water pollution by dyeing industrial wastewater is discharged directly into rivers. Bypass sewers shall promptly be prepared so as to decrease water pollution by dyeing and to avoid flood into the town.
Oita	Oita	Funai Castle Fresh Up Project	More and more domestic wastewater has been discharged into Funai castle internal trench (closed water area), thereby causing serious pollution these days. Treated wastewater shall be supplied to Funai castle internal trench so as to clean the historical remaining.

Table 3 List of idea sewerage FY 1983

Prefecture	City/Town	Project name	Purposes
Akita	Lake Tazawa	Reuse of refuse involving the residents	By using the refused plastic material (vacant bottles) collected through the town cleaning movement as attached media for aerobic contictor the treatment plant construction cost can be cut while promoting effective reuse of such refused material.
Tokyo	Each word/ section	Purification of combined sewer overflows (swirl concentrator)	Wet weather combined sewage drained into rivers and waters shall be utilized to maintain water quality of the public waters after being eliminated the pollatents such as suspended solids therein through the new separation method, that is, the swirl repelator/concentrator.
Kanagawa	Zushi	Energy saving sludge processing using waste heat generated by garbage furnace	The sludge drying facility shall be accommodated with the city garbage furnace: sludge cakes generated from wastewater shall be dried by the waste heat of the furnace. Thus, the amount of sludge can be decreased and stabilized while cutting energy (fuel) and maintenance cost.
Yamanashi	Takane	Aeration-omitting wastewater treatment (Aerobic, anaerobic biofilm process)	The microorganisms which can achieve purification without air have been adapted for treatment thereby omitting aeration, that is, the electric power. This aeration-omitting process is just suitable for small-scale treatment plants.
Toyama	Takaoka	Energy-saving sludge furnace project	Digestion gas generated from the acconpanied excreta disposal facility shall be used as fuel for the sludge furnace and the deodorizing facility equipment thereby saving fuel and cutting maintenance cost.
Shizuoka	Hamamatsu	Simultaneous removal of nitrogen and phosphorus by microorganisms	Eutrophication of Lake Hamana shall be avoided through the removal of nitrogen and phosphorus: Mixed liquor shall go through repetition of stirring and aeration thereby removing nitrogen and phosphorus simultaneously with the help of microorganisms.
Osaka	Osaka	Countermeasure for local flood utilizing the underground of the park - Imazu storage tank	An underground storage tank having capacity about 26,000 m^3 (W 5.0 m x D 5.3m x 109.3 m x two basins, W 10.8 m x D 5.3 m x 115.1 m x two basins) shall be prepared underground of Imazu park in Tsurumi district. The tank shall temporarily store excess stormwater thereby preventing lowlands from flooding.

1.2 Status Quo and Needs for Wastewater Treatment Technology

Five-hundred and twenty-two (522) publicly owned wastewater treatment plants have been in operation by the end of FY 1982. Table 4 shows these plants classified based on their treating methods. This table manifests that the overwhelming majority of treatment plants are adopting the treatment method so called activated sludge process. There is no room to doubt that the activated sludge process is advantageous to the earlier treatment methods. However, the following problems resulting from the situations unique to Japan have been pointed out.

Table 4 Number of plants of each treatment process

(The end of F.Y. 1982)

Treatment process / Planed daily maximum dry weather flow (1,000 m^3/day)	Primary	Intermediate		Secondary							Total
	Sedimentation process	High-rate trickling filter	High-rate aeration	Conventional activated sludge process	Step aeration process	Extended aeration process	Contact stabilization process	Pure oxygen aeration process	Oxidation ditch process	Rotating biological contactor process	
Less than 5	2	2	2	37	5	12	1	2	6	7	76
5 - 10	1	3		45	6	1				3	61
10 - 50	2	6	14	121	13			1		2	158
50 - 100		1	3	69	23					1	96
100 - 500		1		88	21			2			112
More than 500				10	9						19
Total	5	13	19	370	77	13	1	5	6	13	522

Notes: 1. In case two or more treatment processes are employed at a plant, the one treating the largest flow is counted.

2. Details of the total 522 treatment plants:

Municipal sewerage	460
Regional sewerage	41
Special public sewerage	3
Special environment protection municipal sewerage	13

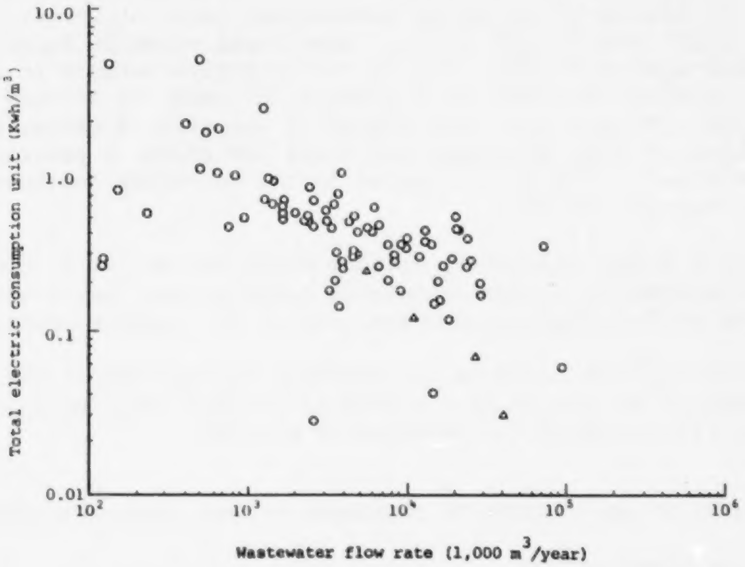


Fig. 3 Electric consumption depending on scale in activated sludge process

The first problem is that the energy efficiency in the activated sludge process is deteriorated as the scale of the treatment plant becomes smaller. Fig. 3 shows electric power required for treatment (including sludge treatment/disposal) of 1 m^3 wastewater depending on the amount of influent to the treatment plant. As mentioned before, sewerage systems are to be spreaded mainly to comparatively small-scale local cities in Japan in the future. With consideration to the limited energy available in the whole nation, the problem of deterioration of the energy efficiency corresponding to small-scalling shall be closed up.

Secondly, sludge disposal holds much more significance in Japan than any other countries since it has only a small land. In fact, lack of room for sludge disposal has naturally popularized the method of expensive sludge incineration and disposal of the outcoming ashes within the site of treatment plant.

In the third place, receiving waters to which the effluent is to be discharged are limited in many cases to rivers and/or inland waters which have only a small dry weather flow. For such cases, advanced treatment to eliminate nutrients is required to avoid eutrophication of rivers and lakes. However, nutrients removal can not be achieved in a stable manner in the activated sludge process. The chemical oxygen demand (COD) is used as one of the effluent standards in some areas. It has turned out that the activated sludge process does not necessarily remove COD to a satisfactory point.

The fourth problem is that the use of treated wastewater is limited due to the remaining refractory substances, color of water, epidemiologic danger, and others. Some local areas in Japan suffer from water shortage every year. One of the effective methods to solve the problem of water shortage is to promote the reuse of treated wastewater. To make the reuse system of the treated wastewater to solve the problem of water shortage with local and seasonal pecuriorities, some technical innovation is needed in the secondary treatment processes before advanced treatment.

As for sludge treatment, various needs can be easily listed: countermeasures for sludge containing heavy metals, improvement of the anaerobic sludge digestion process, and of the compost process, etc.

These problems found in the existing systems and/or the needs recognized to be studied have worked as driving force to start the Biofocus W.T. research and development project.

2. DEVELOPMENT OF NEW WASTEWATER TREATMENT SYSTEMS EMPLOYING BIOTECHNOLOGY

2.1 Outline of Biofocus W.T.

Only ten and several years have passed since the term biotechnology or bioengineering has started to be used. Although these terms have not yet been defined clearly even at present, biotechnology shall be defined, in a wide sense, as follows.

> "Technology to utilize the life maintenance functions of biological organisms for human being"

Biotechnology helps us to benefit our life by utilizing the functions of biological organisms or part of such functions based on the results obtained from the basic studies of vital phenomena. We are employing biological treatment processes for wastewater treatment: that is, it can be said that we have already been utilizing biotechnology in a wide sense. The first and the most important step to apply biotechnology (including the existing and new) for wastewater treatment is to precisely identify the characteristics of the existing problems and classify them based on the methods for solution.

The basic strategy of the Biofocus W.T. begins by classifying the existing problems of wastewater treatment as shown in Fig. 4. The problems are identified to be classified as follows:

(1) The current technology is not economical.

(2) The current technology is not stable nor effective.

(3) The problem cannot be solved by the current technology.

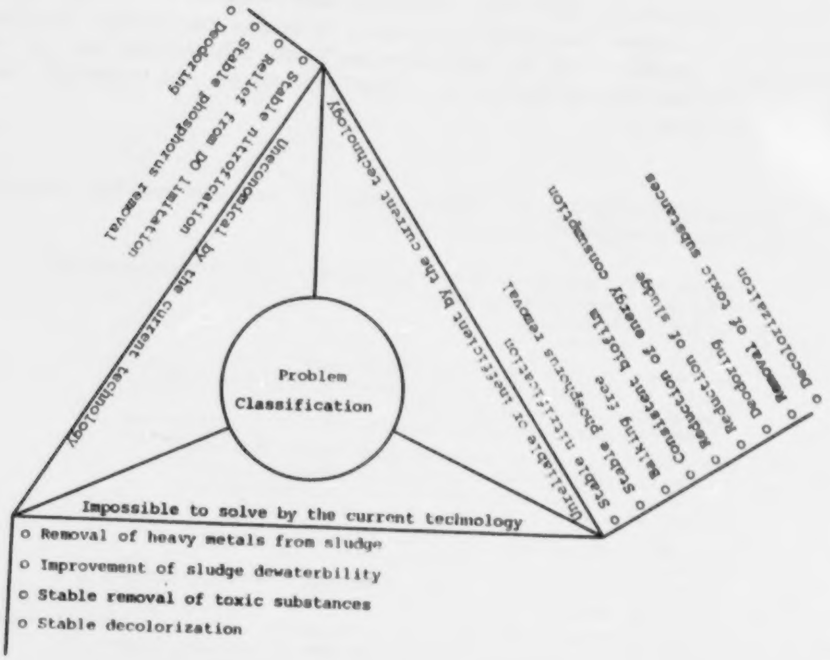

o Deodoring
o Stable phosphorus removal
o Relief from DO limitation
o Stable nitrofication
Uneconomical by the current technology
Unreliable or inefficient by the current technology
Problem Classification
o Stable nitrification
o Stable phosphorus removal
o Bulking free
o Consistent biofilm
o Reduction of energy consumption
o Reduction of sludge
o Deodoring
o Removal of toxic substances
o Decolorization
Impossible to solve by the current technology
o Removal of heavy metals from sludge
o Improvement of sludge dewaterbility
o Stable removal of toxic substances
o Stable decolorization

Fig. 4 Problems in current wastewater treatment technologies and their classifications

Along with classification of the problems, the methods for solution can also be classified as follows:

(1) Improvement of an existing treatment technology

(2) Application of a technology from a different field (technology transfer)

(3) Search for novel microorganisms not prently utilized in treatment and development of the application methods thereof

(4) Improvement of microorganisms by genetic alteration

The definition of the existing technologies differs from country to country. Such definition by field engineers and that by scholars do not necessarily conform to each other within Japan. Provided that the existing technologies are to be understood as those which went through historical and spatial selection and those which have been practically and widely adopted, the following can be listed: the activated sludge process as a suspended growth type of biological wastewater treatment process, the trickling filter and the rotating biological contactor as a biofilm process, and the aerobic and anaerobic sludge digestion as suspended growth types of sludge treatment process.

The Biofocus W.T. project, leaving improvement of these existing technologies to other research projects, focuses on study, development, and practical application of solution methods classified as (2) through (4) in the forementioned category. Table 5 shows the research subjects of the Biofocus W.T.

Table 5 Research subjects of development of new wastewater treatment systems employing biotechnology

I. Development of immobilization technology of enzymes and microorganisms
  - Study on immobilization methods
  - Study on bioreactor
  - Study on biosensor

II. Development of enzyme and microorganism bank
  - Study on separation and classification of microorganisms
  - Study on evaluation of enzymes
  - Study on preservation and mass cultivation of microorganisms

III. Development of novel bacteria by using genetic alteration
  - Study on needs and feasibility of the genetic control
  - Improvement of novel microorganisms by recombinant DNA
  - Development of novel microorganisms by cell fusion

IV. Development of combined technology of physical/chemical Processes with biological processes
  - Study on evaluation for physical/chemical treatment processes
  - Study on total treatment system

V. Application of new technologies to the practical wastewater treatment system
  - Application to the publicly owned wastewater treatment system
  - Application to the single house wastewater treatment

As for the first subject, "Development of immobilization technology of enzymes and microorganisms", the technology which has already been practiced in the fermentation industry and the pharmaceutical industry shall be transferred to the wastewater treatment field. The fluidized biological reactor utilizing fine sand or granular activated carbon has been wordly studied and developed. In this subject, the carrier themselves used in such a carrier bound method shall first be developed. The entrapped immobilization method shown in Fig. 5 shall also be studied.

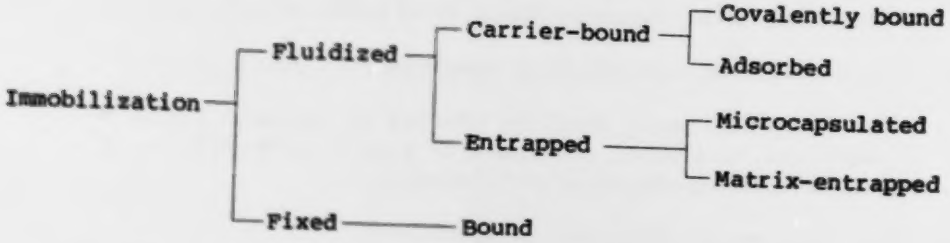


Fig. 5 Classification of immobilization methods

For the biosensor technology, which is one of the application technologies of enzyme immobilization, development shall be done for more highly stable sensor for water quality monitoring.

The purpose of the second subject, "Development of enzyme and microorganism bank", is to collect and study unused microorganisms as well as to study the application method for wastewater treatment. Natural selection of microorganisms, especially those living in the extreme environments, is already initiated. As for the bacteria known to have specific abilities such as nitrifying bacteria, methane bacteria, and photosynthetic bacteria, the mass cultivation methods with the help of immobilization technology are now being studied.

The third subject, "Development of novel bacteria by using genetic alteration", requires to reveal genetic codes responsible for producing specific proteins, or enzymes, in the first place. Much efforts are to be intensified in the selection of mutants, because the use of novel bacteria produced by recombinant DNA is stricly limited to the reactor with the capacity less than 2,000 litters.

Those three subjects above mentioned are all related to the use of microorganisms to the treatment of wastewater. However, wastewater treatment consists of not only biochemical processes but also physical and/or chemical processes. The forth and fifth subjects are aiming at the development of total wastewater treatment system followed by demonstrative verification in the fields.

2.2 Joint Research System

The Biofocus W.T. is expected to play a kind of revolutionary role in the wastewater treatment technology for the next century. The project cannot be achieved only by the Ministry of Construction without seeing the limit: the project shall be promoted by several joint researches by the Ministry of Construction, municipal bodies, public corporations, and private enterprises. Therefore, the "Regulations on Joint Study of the Institutes of Ministry on Construction" was revised in June, 1985. Primary revision was that the Institutes became able to carry out the joint research with a private company.

Methods of joint research shall be classified into two:

(1) The Ministry of construction specifies the joint party:

The joint party shall be limited to national research institutes, municipal governments, public corporations, or associations representing the industry.

(2) The joint party shall not be specified:

The Director General of the Public Works Research Institute or Building Research Institute of the Ministry of Construction shall offer for public subscription for the joint party. The applied enterprises shall be judged and selected in the committee board with consideration to their ability to perform the study.

As one of the characteristics of the joint research, the priority is given to the joint party to own the right to use the results thereof, especially the patents, for no more than five years from the completion date of the joint research. It is true that there were some refutation against this way saying that it stirred up competition among enterprises in vain. On the other hand, many enterprises welcome this way understanding that the Ministry of Construction would like to reflect the benefits of excellent technologies to the public works.

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Tenth United States/Japan Conference
on Sewage Treatment Technology

AUTOGENOUS COMBUSTION OF SEWAGE SLUDGE

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1. INTRODUCTION

The standard flow of a sewage sludge incinerator is shown in Fig. 1. The flow can be broadly classified into two parts, namely, the incinerator main body and exhaust gas treatment.

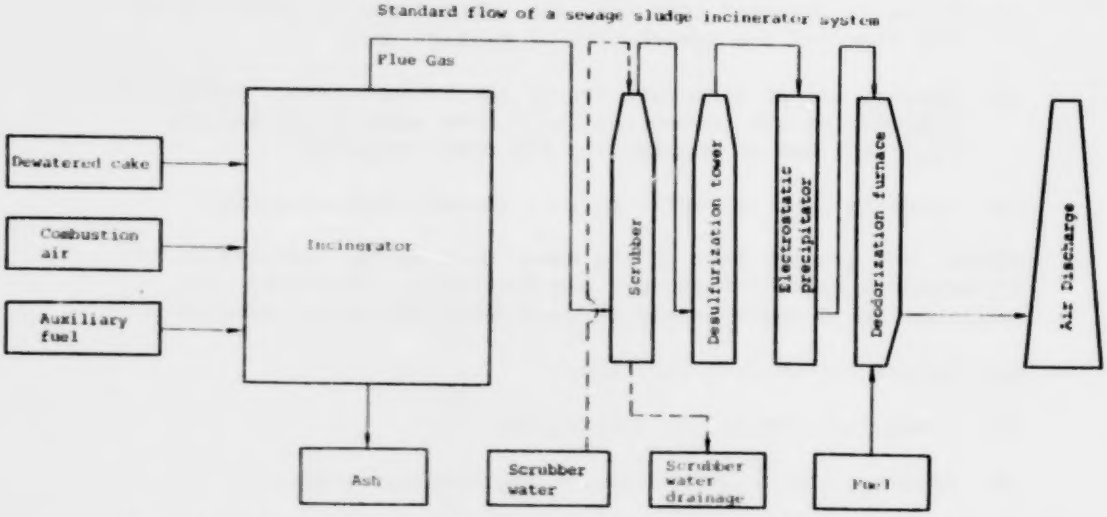

Standard flow of a sewage sludge incinerator system
Flue Gas
Dewatered cake
Combustion air
Auxiliary fuel
Incinerator
Scrubber
Desulfurization tower
Electrostatic precipitator
Deodorization furnace
Air Discharge
Ash
Scrubber water
Scrubber water drainage
Fuel

Fig. 1 Standard flow of a sewage sludge incinerator system

Main items reviewed in this paper for energy-conservation are as follows:

(a) Energy-conservation measures from the viewpoint of the material balance of an overall incinerating process.

(b) Energy-conservation measures from the viewpoint of the heat balance of an overall incinerating process.

In regard to the material balance, factors affecting energy balance from a viewpoint of input and exhausted materials are discussed as follows.

(A) Fuel: Heavy oil A or kerosene are used. In some treatment plants digesting sewage sludge, however, digester gas is used as an auxiliary fuel.

In a standard incinerator, fuel is used as an auxiliary source of heat for burning cake that does not easily burn, and for a exhaust gas deodorization device (thermal deodorization method). The consumption of auxiliary fuel varies depending on the characteristics of the input cake as fuel (moisture content and heating value), temperature, and amount of exhaust gas.

Consequently, the following four measures may be considered to save fuel.

(a) Improve the characteristics of cake as a fuel, that is, reduce the cake's water content and improve its constituents to increase the heating value.

(b) Minimize the heat loss with the exhaust gas by reduction of the flow and the temperature of exhaust gas.

(c) Provide for an effective use of the exhaust heat. Among such methods are the recovery of heat from exhaust gas and the recurrent use of cooling air for other purposes.

(d) Increase the fuel-efficiency in thermal deodorization.

(B) Water: The greater part of the water used by the incinerator is for scrubber water for exhaust gas treatment. Therefore, the following three measures can be considered for water efficiency.

(a) Reduce the exhaust gas flow.

(b) Lower the exhaust gas temperature.

(c) Increase the recirculation of the scrubber water.

(C) Chemicals: The majority of chemicals are for use in desulfurization. Because the consumption of chemicals varies depending on the dewatered cake's sulfide content, chemical aid for dewatering should contain the least amount of sulfide.

(D) Power: The greater part of power is used for blowers and pumps. Therefore, two measures can be considered to reduce power consumption.

(a) Reduce the exhaust gas flow.

(b) Lower the exhaust gas temperature.

Taking into account items (A) to (D), the following factors need to be considered as energy-saving measures for the incinerator.

a) improve the dewatered cake's fuel characteristics.

b) Reduce the exhaust gas flow and lower its temperature.

c) Increase the efficiency of heat recovery.

d) Reduce the energy used for the exhaust gas deodorization.

e) Improve the efficiency in combustion

The following feasibility studies were made to determine how these factors can be incorporated into an incinerator and included investigations of the actual incinerators in use.

2. OPERATING STATUS OF INCINERATING FACILITIES

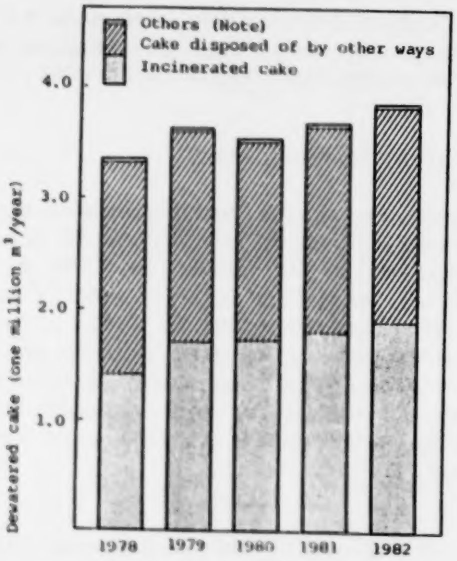
The total volume of sewage sludge being disposed of in Japan in 1982 was 2,480,000 (m^3/year), in which the dewatered cake accounts for about 80% and the incinerated ash for about 10%. The majority of sludge is disposed of for landfill, including inland landfill and coastal reclamation, which accounted for about 70% in 1982.

The potential sludge disposal site must satisfy the basic conditions shown in Table 1 in Japan where land is highly utilized. Therefore, there has been a tendency for each sewerage authority to adapt an incinerating system which makes it easier to meet the conditions shown in Table 1.

Table 1 Basic conditions of sewage disposal site

Broader classification	Middle classification	Contents of basic conditions
Technical requirements	Requirements for the sludge disposal site	o The site must have enough space to be disposed of planned quality of sludge. o The ground must be safe after it is landfilled.
	The quantity and property of sludge to be disposed	o A step-by-step disposal plan must be clearly stated. o The property of sewage sludge must comply with the disposal standards.
Regional conditions	Acquisition of site	o The consent from the parties with rights to the land needs to be secured.
	Land use	o The potential site must conform to the regulations concerning the use of the land. o A cencrete plan to utilize the site after landfill must be formulated in accordance with city planning.
Economic conditions	Final disposal site	o The construction and maintenance costs of the disposal site must be economical (dam, dike, or shore protection works, sealing walls and wastewater treatment facilities).
	Transportation	o The shortest possible transportation distance is desirable, and the transportation method needs to be optimized.
Environmental conditions	Surrounding environment	o Proper measures to counter noise, odor, and scattering can be taken when performing disposal work. o The disposal work should not adversely affect the use of surrounding land.

Fig. 2 clearly shows the increase in sewage treatment facilities with incinerating system. As shown in Fig. 2, the volume of cake being incinerated has continuously increased, reaching 1,900,000 (m^3/year) in 1982. The dewatered cake being incinerated accounted for 50% of the total volume of cake produced.



(Note) Sludge disposed of in the form of liquid raw sludge or digested sludge expressed in terms of dewatered cake

Fig. 2 The treated and disposed amount of dewatered cake

The following types of incinerators are used in Japan for incinerating sewage sludge:

(i) Multiple-hearth furnace
(ii) Fludize bed furnace
(iii) Rotary kiln
(iv) Stoke furnace

The numbers of incinerators in operation, classified by types, are shown in Fig. 3. The increase in the number of fluidized bed furnaces are remarkable.

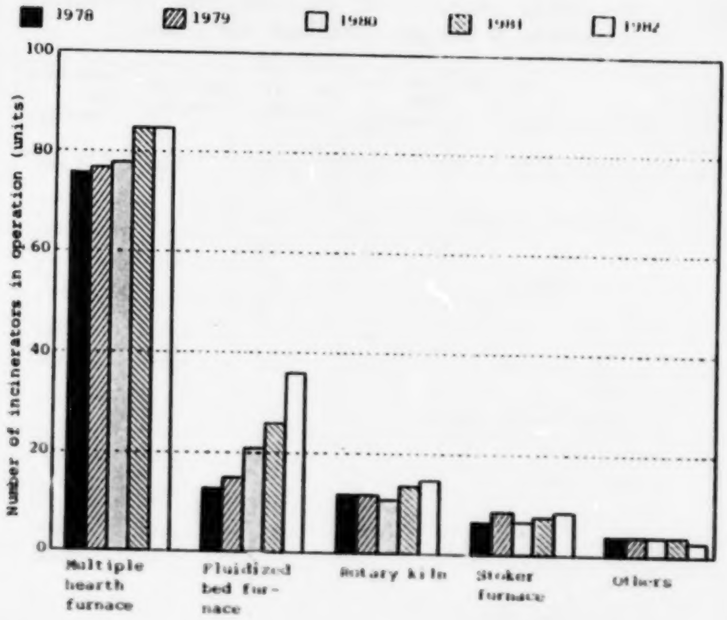


Fig. 3 Number of incinerators in operation classified by types of furnaces

Table 2 shows the summation of incinerators' treating capacities (capacity x units) in terms of wet cake. Among the total number of incinerators (148 units) in operation in 1982, the multiple-hearth furnace accounted for 57%, and fluidized-bed furnaces accounted for 24%. On the capacity base, the total capacity of the incinerators in 1982 was 10,600 ton/day (in terms of the wet cake), in which the multiple-hearth furnaces accounted for 79% and the fluidized bed furnaces accounted 16%.

Table 2 The total capacity of incinerators classified by size (capacity x units) (ton wet cake/day)

Fiscal Year	Capacity Type of furnace	Less than 100 (ton/day)	10 - 50 (ton/day)	50 - 100 (ton/day)	More than 100 (ton/day)	Total
1978	Multiple-hearth furnace	31	1,131	960	4,840	6,920
	Fluidized-bed furnace	3	460	0	0	463
	Rotary kiln	40	139	0	0	179
	Stoker furnace	8	207	0	0	215
	Others	8	35	55	0	98
	Total	90	1,972	1,015	4,840	7,917
1979	Multiple-hearth furnace	31	1,147	1,030	5,140	7,348
	Fluidized-bed furnace	3	555	0	0	558
	Rotary kiln	40	127	0	0	167
	Stoker furnace	8	277	0	0	285
	Others	8	48	55	0	111
	Total	90	2,154	1,085	5,140	8,469
1980	Multiple-hearth furnace	22	1,147	1,150	5,440	7,759
	Fluidized-bed furnace	3	772	60	0	835
	Rotary kiln	40	78	53	0	171
	Stoker furnace	8	207	0	0	215
	Others	8	48	55	0	111
	Total	81	2.252	1,318	5,440	9,091
1981	Multiple-hearth furnace	22	1,302	1,210	5,790	8,325
	Fluidized-bed furnace	6	820	320	0	1,146
	Rotary kiln	64	78	53	0	195
	Stoker furnace	8	257	0	0	265
	Others	8	48	55	0	111
	Total	108	2,506	1,638	5,790	10,042
1982	Multiple-hearth furnace	22	1,303	1,210	5,790	8,325
	Fluidized-bed furnace	9	968	670	0	1,647
	Rotary kiln	72	78	53	0	203
	Stoker furnace	8	271	0	0	279
	Others	0	48	55	0	103
	Total	111	2,668	1,988	5,790	10,557

The average capacity per unit of the multiple-hearth furnace is about 98 (ton/day) and the fluidized-bed furnace about 46 (ton/day). The maximum capacity of the multiple-hearth furnace per unit is 300 (ton/day).

Recently, the construction of a fluidized bed furnace with large capacity has become possible and incinerators with a capacity of more than 200 (ton/day) are being constructed. The marked increase in the number of fluidized-bed furnaces after 1978 was because of the following reasons:

(1) Finding landfill sites has become difficult even in medium-sized cities, resulting in the need for incinerators with a small capacities.

(2) The intermittent operations of a fluidized bed furnace is relatively easy compared with the multiple-hearth furnace.

(3) Fluidized-bed furnaces do not require particular measures for deodorization because the odor of the exhaust gas is much weaker than that of a multiple-hearth furnace.

Table 3 shows the number of sewage treatment plants with incinerators classified by the sludge treatment system. At the end of 1982, the total number of sewage treatment plants in operation reached 529, in which 89 had incinerators accounting for 17% of the treatment plants.

Table 3 Installation status of incinerators classified by the sludge treatment system (1982)

Sludge treatment process Type of furnace	Direct dewatering	Digestion and dewatering	Heat treatment and dewatering	Others	Furnace not in operation	Total
Multiple-hearth furnace	28	13	1	0	5	47
Fluidized-bed furnace	19	5	0	1	1	26
Rotary kiln	3	1	0	0	0	4
Stoker furnace	0	0	5	1	1	7
Others	1	1	0	0	0	2
Combination use	0	1	1	1	0	3
Total	51	21	7	3	7	89

(Note) 1. "Combination use" refers to the treatment plants that use more than two different types of incinerators.
2. "Others" in the sludge treatment process refers to the treatment plants with CG process or those with only sludge treatment process.

The greater part of sewage treatment plants equipped with an incinerator(s) employ raw sludge dewatering. In 1982, such sewage treatment plants accounted for 51, or 62%, of the total of 82 treatment plants equipped with an incinerator(s).

The fuel consumption of incinerators is shown in Table 4. As seen in this table, the incinerator consumes 48.7ℓ of heavy oil per 1 m^3 of wet cake on the average. Improving the facilities that consume such a great deal of energy has become an urgent problem in energy conservation for the sewage treatment plants.

Table 4 Quantity of the incinerated sludge and the fuel consumption

Fiscal year	1979	1980	1981	1982
Consumption	85,051 m^3	81,766 m^3	86,758 m^3	91,684 m^3
Quantity of incinerated cake	1,654,620 m^3	1,592,830 m^3	1,730,550 m^3	1,881,270 m^3
Fuel consumption per 1 (m^3) of cake	51.4 ℓ/m^3	51.3 ℓ/m^3	50.1 ℓ/m^3	48.7 ℓ/m^3
Number of treatment plants	64	63	65	77

3. HEAT CHARACTERISTICS OF DEWATERED CAKE

Because of the change in the quality of sewage and improvement in sludge conditioning method, and because of advances of the dewatering machines, the property of the dewatered cake has been significantly changed. The following is the result of investigations made on the heat characteristics of dewatered cake.

3.1 Outline of Investigations

The heat characteristics of dewatered cake can be affected greatly by the cake's water content, volatile solids and the element composition of solids. Therefore, these properties and higher heating value of the cakes collected from 59 sewage treatment plants were measured to grasp the heat characteristics of the cake. As shown in Table 5-(1), the samples were taken from wide variety of treatment plants from view-points of the collection system, the kinds of sludge supplied to the dewatering process (digested sludge or thickened raw sludge), and the dewatering machines. The measured items and the methods of measurement are as shown in Table 5-(2).

Table 5-(1) Kind and number of samples

kind of dehydrator	Collection system	Kind of sludge	Number of samples	Total (1)	Total (2)	Grand total
Centrifuge	Combined sewer	Digested sludge	7	9	15	59
		Mixed raw sludge	2			
	Separate sewer	Digested sludge	2	6		
		Mixed raw sludge	4			
Belt press	Combined sewer	Digested sludge	4	7	17	
		Mixed raw sludge	3			
	Separate sewer	Digested sludge	4	10		
		Mixed raw sludge	6			
Vacuum filter	Combined sewer	Digested sludge	4	8	14	
		Mixed raw sludge	4			
	Separate sewer	Digested sludge	2	6		
		Mixed raw sludge	4			
Filter press	Combined sewer	Digested sludge	2	6	12	
		Mixed raw sludge	4			
	Separate sewer	Digested sludge	3	6		
		Mixed raw sludge	3			
Screw press	Separate sewer	Mixed raw sludge	1	1	1	

Table 5-(2) Items of analysis and analytical methods

Item of analysis	Analytical method
Moisture content, ignition loss, higher heating value	Test method for sewage
Carbon, hydrogen, nitrogen	CHN automatic analyzer
Sulphur	JIS-M-8813
Oxygen	Ignition loss - (C+H+N+S)

3.2 Results of Measurement

(1) Heating value of the cake

Fig. 4 shows a summary of the relations between the higher heating value (H_h) and the organic contents (V) classified by the dewatered cakes. The sludge cakes are classified into two categories: one is the sludge dewatered by a centrifuge or a belt press with polymer addition (hereinafter called polymer cake), and the other is the sludge dewatered by a vacuum filter or a filter press with lime and ferric chloride addition (hereinafter called lime cake).

Equations 1 and 2 show the regression equations between the higher heating value and the organic content, expressed by ignition loss, for polymer cake and lime cake, respectively. In either case, a good correlation existed, and the higher heating value could be expressed as a function of the organic content.

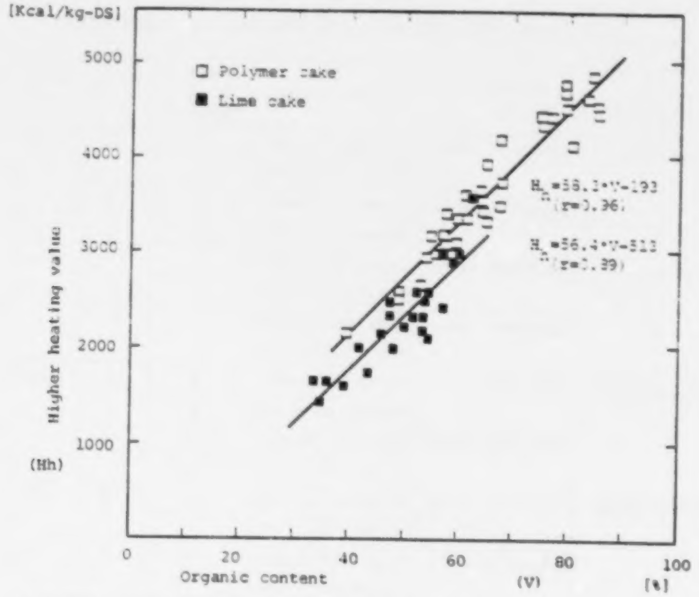


Fig. 4 Relationship between the higher heating value of dewatered cake and organic content

Polymer cake
H_h = 58.3 V - 193 .... Equation 1

Lime cake
H_h = 56.4 - 513 .... Equation 2

In which, H_h is the higher heating value in Kcal/kg-DS and V is the organic content expressed by ignition loss in %. The higher heating value of lime cake is low compared with that of polymer cake even if the organic content is the same, and the differences are 450 to 570 (kcal/kg-DS) within the range of organic contents from 40% to 70%. The following factors are considered to be the causes of these differences.

(a) When the inorganic chemicals (lime and ferric chrolide) are added, these chemicals remain in the cake mainly in the form of lime {$Ca(OH)_2$} and ferric hydroxide {$Fe(OH)_3$}. When the ignition loss of the cake is measured, water from these compounds evaporates according to the reactions shown by equations 3 and 4, resulting in a greater ignition loss than that of the original sludge.

(b) As shown by Equation 4, the reaction exhibits a heat absorption of about 350 (Kcal/kg lime)

$2Fe(OH)_3 \quad Fe_2O_3 + 3H_2O$ .... Equation 3

$Ca(OH)_2 \quad CaO + H_2O - 355$ (Kcal/kg lime) ..... Equation 4

The reason of the smaller correlation coefficient for lime cake than that for polymer cake is considered to be attributable to the difference in the dose rate of the coagulant.

Equations 5 and 6 show the relations between the lower heating values and the organic contents of the cakes. As in the case of the higher heating values, the relationship between the lower heating values of polymer cake and lime cake can be expressed by linear functions of volatile solids as follows:

Polymer cake
$H_\ell = 54.0\ V - 150\ (r = 0.96)$ .... Equation 5

Lime cake
$H_\ell = 51.2\ V - 458\ (r = 0.87)$ .... Equation 6

In which, H_ℓ is the lower heating value of the cake in Kcal/kg-DS.

Fig. 5 shows the relationship between the higher and lower heating values of the cakes. The correlation between them is quite high. The lower heating values of the cakes are 0.93 times as much as the higher heating values for both polymer and lime cakes.

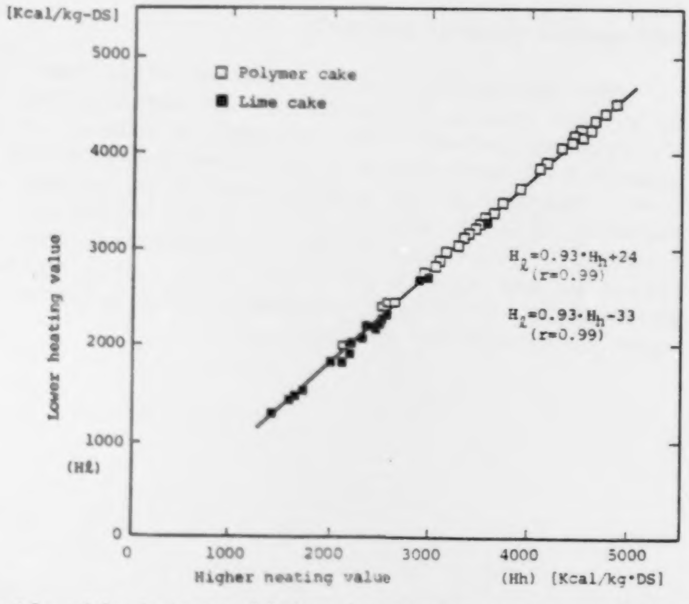


Fig. 5 Relationship between higher heating value and lower heating value

(2) Element composition of solids

As mentioned, the heating value of a cake is closely related to the organic content. The relationship between the element composition (carbon, oxygen, hydrogen, and sulphur) of solids and the organic content was examined. Fig. 6 shows the relationship between the content of each element and the organic content of the cake.

(a) Carbon content

The carbon content of lime cake is less by 3% to 5% than that of polymer cake within the range of organic content of 40 - 70%.

(b) Oxygen content

The oxygen content of lime cake is higher by 4% to 7% than that of polymer cake.

(c) Hydrogen content

The hydrogen content of lime cake is higher by 0.2% to 0.4% than that of polymer cake.

Both the hydrogen and oxygen contents exhibit different tendencies from the carbon content, and the lime cake has higher hydrogen and oxygen contents than the polymer cake. These are attributable to hydrogen and oxygen contained in hydroxides, which is lost as water during the measurement of ignition loss as shown in Equations 3 and 4.

(d) Inflammable sulphur content

The inflammable sulphur content of polymer cake was found to be less than 1.0% and to have no definite relationship with the organic content. The inflammable sulphur, however, is an element that accounts only for a small percentage of ignition loss. Therefore, the sulphur content of polymer cake was regarded as being about 1.0% of total solids for reasons of simplicity. Conversely, the inflammable sulphur content of lime cake was found practically zero. This may be attributable to calcium sulfate ($CaSO_4$) produced by the reaction of sulphur and lime added.

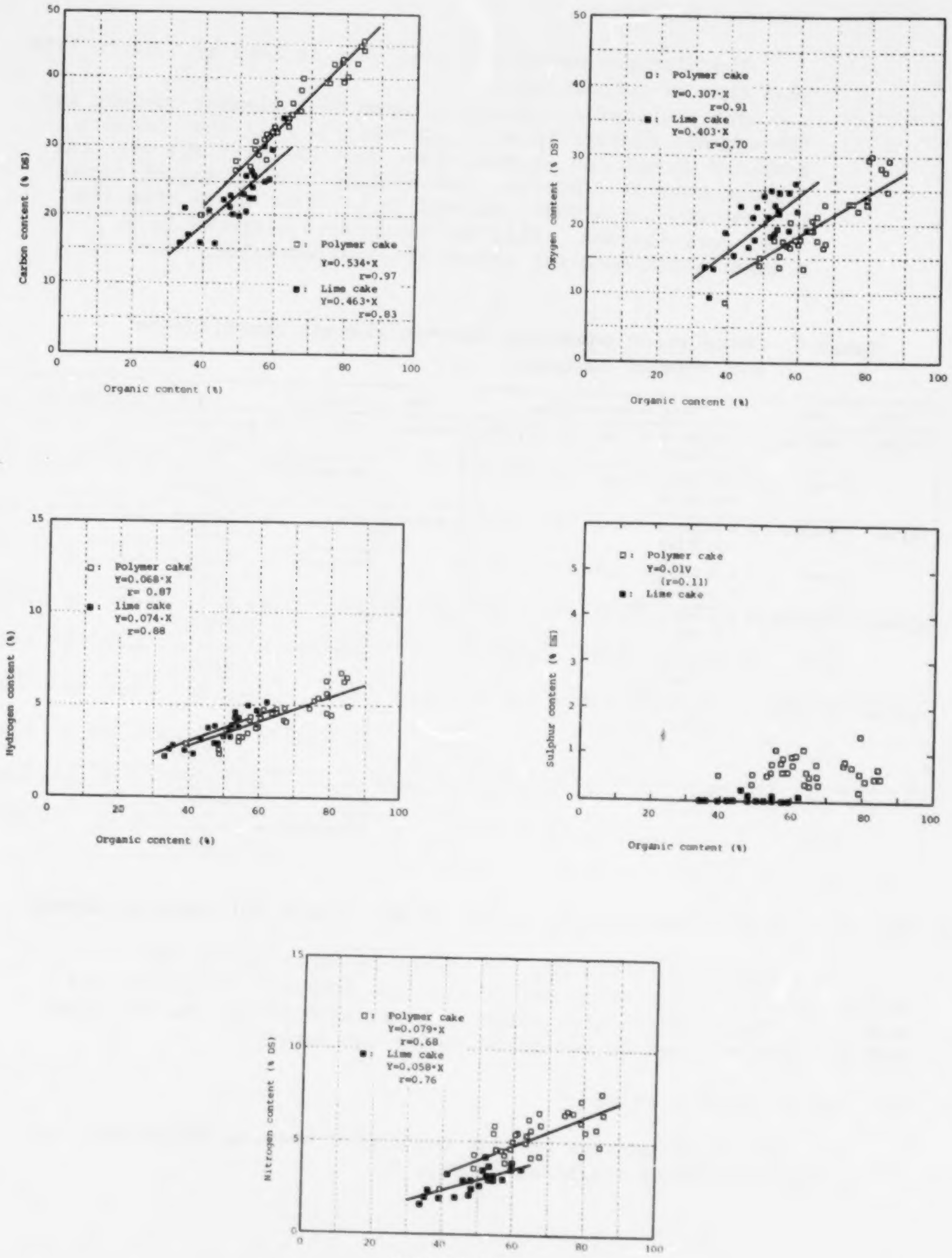


Fig. 6 Relationship between organic content and the others

(e) Nitrogen content

The nitrogen content of lime cake is less by 0.8% to 1.5% than that of polymer cake.

The regression equation between each element content and the organic content is shown in Table 6. All the elements examined except for sulphur, have good correlations with the organic content. However, correlation between the nitrogen content and the organic content is a little worse than the other correlations. This may be because nitrogen is an element which does not affect the heating value.

Table 6 Correlation equations between element constituents and organic content

Item	Cake	Correlation equation
Carbon	Polymer	$C = 0.498 \cdot V + 2.46$ $(r = 0.97)$ $= 0.534 \cdot V$
	Lime	$C = 0.479 \cdot V = 0.76$ $(R = 0.83)$ $= 0.463 \cdot V$
Oxygen	Polymer	$O = 0.387 \cdot V = 5.44$ $(r = 0.91)$ $= 0.307 \cdot V$
	Lime	$O = 0.360 \cdot V + 2.20$ $(r = 0.70)$ $= 0.403 \cdot V$
Hydrogen	Polymer	$H = 0.079 \cdot V = 0.79$ $(r = 0.87)$ $= 0.068 \cdot V$
	Lime	$H = 0.095 \cdot V - 1.07$ $(r = 0.88)$ $= 0.074 \cdot V$
Sulphur	Polymer	$S = -0.002 \cdot V + 0.813$ $(r = 0.11)$ $= 0.01 \cdot V$
	Lime	- -
Nitrogen	Polymer	$N = 0.067 \cdot V + 0.87$ $(r = 0.68)$ $= 0.079 \cdot V$
	Lime	$N = 0.066 \cdot V - 0.39$ $(r = 0.76)$ $= 0.058 \cdot V$

Item	Cake	Correlation equation
Carbon	Raw	$C = 0.495 \cdot V + 2.73$ $(r = 0.95)$ $= 0.533 \cdot V$
	Digested	$C = 0.501 \cdot V + 2.29$ $(r = 0.97)$ $= 0.539 \cdot V$
Oxygen	Raw	$O = 0.382 \cdot V - 5.09$ $(r = 0.85)$ $= 0.311 \cdot V$
	Digested	$O = 0.389 \cdot V - 5.58$ $(r = 0.91)$ $= 0.302 \cdot V$
Hydrogen	Raw	$H = 0.08 \cdot V - 0.745$ $(r = 0.80)$ $= 0.070 \cdot V$
	Digested	$H = 0.073 \cdot V - 0.435$ $(r = 0.88)$ $= 0.065 \cdot V$
Sulphur	Raw	$S = 0.005 \cdot V + 0.88$ $(r = -0.23)$ $= 0.007 \cdot V$
	Digested	$S = 0.0013 \cdot V + 0.009$ $(r = 0.56)$ $= 0.013 \cdot V$
Nitrogen	Raw	$N = 0.049 \cdot V + 2.15$ $(r = 0.48)$ $= 0.079 \cdot V$
	Digested	$N = 0.078 \cdot V + 0.135$ $(r = 0.74)$ $= 0.080 \cdot V$

3.3 The Difference in the Heating Values of Raw Sludge and Digested Sludge

The difference in the heating values of cakes due to the differences in sludge being supplied to the dehydrating process was examined. The polymer cake, which is least-affected by the coagulant addition was selected as the object under the survey.

(1) Higher heating value

The relationships between the higher heating values and the organic contents are shown in Fig. 7.

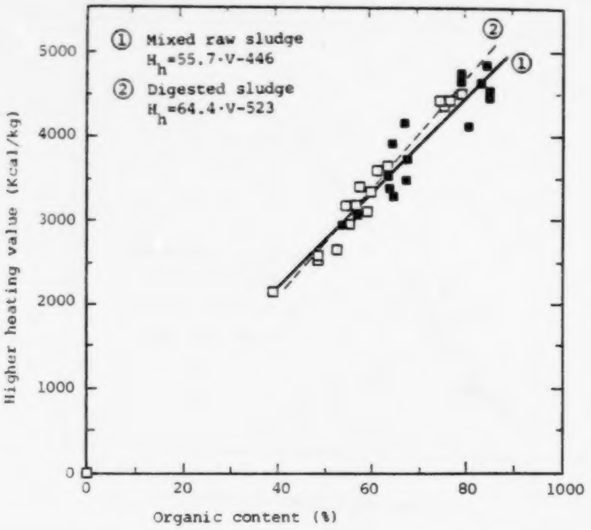


Fig. 7 Relationship between organic content and higher heating value

The regression equations for raw sludge cake and digested sludge cake are expressed by Equations 7 and 8, respectively.

The differences in the higher heating values of the raw and the digested sludge cakes are in the range of -40 to +220 (Kcal/kg-DS) and are not so large within the range of organic contents from 50% to 80%.

Thickened Sludge

$H_h = 55.7\ V - 466\ (r = 0.92)$ .... Equation 7

Digested Sludge

$H_h = 64.7\ V - 523\ (r = 0.98)$ .... Equation 8

(2) Element composition

The regression equation between the content of each element and the organic content expressed by ignition loss is shown in Table 6. The difference between thickened raw sludge and digested sludge is small. Therefore, the higher heating value can be computed from the organic content by using the respective equation mentioned above, regardless the type of feed sludge.

Sulphur in solids can be separated into inflammable sulphur and noninflammable sulphur. Fig. 8 shows the relationship between inflammable sulphur and total sulphur. Only very little difference exists between the total sulphur contents in the raw sludge and the digested sludge. Total sulphur is about 1.5 times greater than inflammable sulphur.

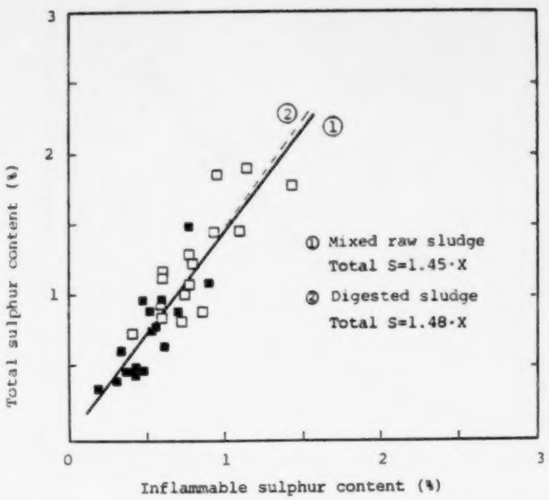


Fig. 8 Relationship between total sulphur and inflammable sulphur

3.4 The Usable Heating Value ($H_{\ell(w)}$) of Wet Cake

Cake differs from common fuel because it contains a great deal of water. Therefore, when an evaluation of cake as fuel is attempted, moisture content of the cake must be considered. In such a case, the usable heating value is generally taken to be an index, which is computed by using Equation 9.

$$H_{\ell(w)} = H_{\ell} \left(1 - \frac{w}{100}\right) = 600 \frac{w}{100} \quad \text{.............................} \quad \text{Equation 9}$$

H_{ℓ}; Lower heating value based on dry solids
w; Moisture content of cake

A summary of the usable heating values of various types of cakes expressed in terms of wet cake is shown in Table 7.

Table 7 Difference in the usable heating value depending on the type of dehydrator

Dehydrator		Moisture content (%)	Organic content (%)	Usable heating value	
				(Kcal/ kg-DS)	Kcal/ kg-wet cake)
Centrifuge	Average	80.5	61.5	3,090	114
	Range	77.4 - 84.8	39.4 - 80.3	2,000 - 4,130	-69 - 506
Belt press	Average	80.7	69.3	3,650	218
	Range	73.0 - 84.5	53.1 - 84.6	2,450 - 4,420	-59 - 606
Vacuum filter	Average	77.6	49.7	2,150	15
	Range	73.3 - 83.2	53.5 - 62.4	1,440 - 3,300	-132 - 375
Filter press	Average	59.2	50.2	2,040	477
	Range	47.3 - 66.9	35.1 - 58.9	1,550 - 2,690	324 - 677

As seen from this table, the usable heating value of the cake obtained by means of a filter press showed the highest average value of 480 (Kcal/kg - wet cake), followed by 220 (Kcal/kg - wet cake) of the cake from a belt-press filter. The cake obtained by a vacuum filter exhibited the lowest value of 20 (Kcal/kg - wet cake). As shown by Equations 5 and 6, the lower heating values based on dry solids can be expressed as the functions of the organic content. Therefore, the usable heating values of both polymer and lime cakes can be expressed by Equations 10 and 11, respectively.

Polymer cake

$$H_{\ell(w)} = (54.0\ V - 150)\ (1 - \frac{w}{100}) - 6w \quad \text{..................... Equation 10}$$

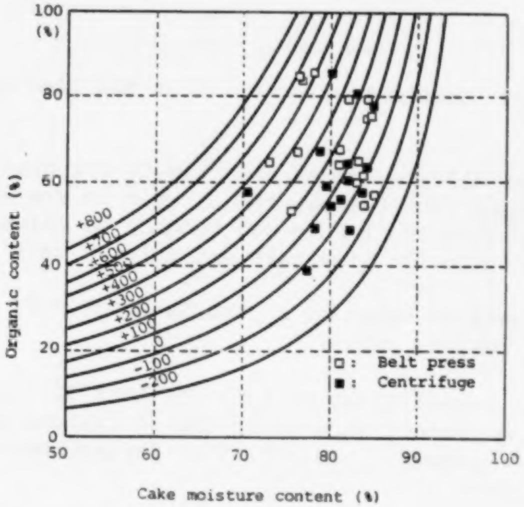


Fig. 9 Usable heating value of the polymer cake

Inorganic cake

$$H_{\ell(w)} = (51.2\ V - 458)\ (1 - \frac{W}{100}) - 6W \quad \text{..................... Equation 11}$$

When the moisture content and the organic content of the cake are the same, the usable heating value of polymer cake is higher than that of lime cake. With regard to the usable heating value, the polymer cake with moisture content of 80% and the organic content of 70%, is equivalent to the lime cake with about 3% less moisture content and 10% more organic content. The conclusions can be summarized as follows:

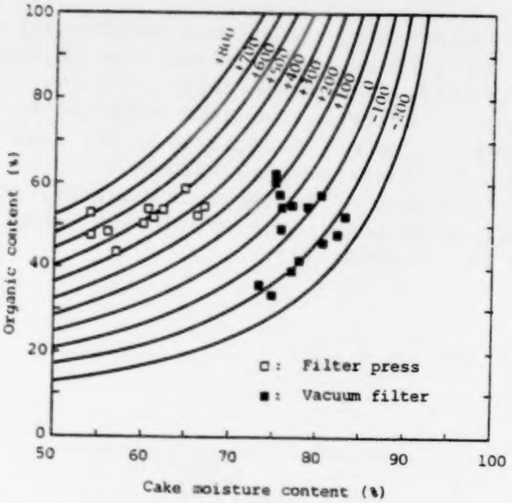


Fig. 10 Usable heating value of the lime cake

(a) No essential difference was observed in the heating value and the element composition between the cakes from raw sludge and digested sludge. The heating values can roughly be calculated by the organic contents for lime cake and polymer cake respectively.

(b) The lower heating value of a cake is about 0.93 times as much as the higher heating value.

(c) The element compositions of ignition loss of the fed sludge to the dehyderating process were about 53% of carbon, about 31% of oxygen, about 7% of hydrogen, about 1.0% of inflammable sulphur, and about 7% of nitrogen.

(d) Among the dehyderating machines, the filter press can produce the cake with the highest heating value, followed by the belt press filter, centrifuge, and vacuum filter.

4. EVALUATION OF MULTIPLE HEARTH FURNACE FOR ENERGY SAVING

The two types of multiple hearth furnaces (Model II and Model III) that have been improved with respect to heat recovery are compared with a conventional furnace (Model I) putting particular emphasis on heat balance.

4.1 Incinerator Models

The following incinerators were selected as the subjects for discussion.

[Model I] A conventional type of incinerator. The air used to cool the incinerator shaft is discharged to the atmosphere without recovering any heat. The heat is not recovered from the exhaust gas either.

[Model II] The air used to cool the shaft is designed to be circulated into the furnace to recover the heat. The heat is not recovered from the exhaust gas.

[Model III] The shaft-cooling air is circulated to recover the heat, and the heat is also recovered from the exhaust gas. (Although not calculated with the model, the unburned materials and lost materials can also be reburned).

4.2 General Equations of Mass Balance and Heat Balance

First, the basic conditions related to input and output of materials were determined, and the equations of mass and heat balances were formulated. Next, the equation was modified to calculate the deficit of heat which gave the required amount of auxiliary fuel.

(1) Heat balance

The configurations of the three model incinerators illustrated in Fig. 11 can be expressed with respect to material balance in a block diagram in Fig. 12.

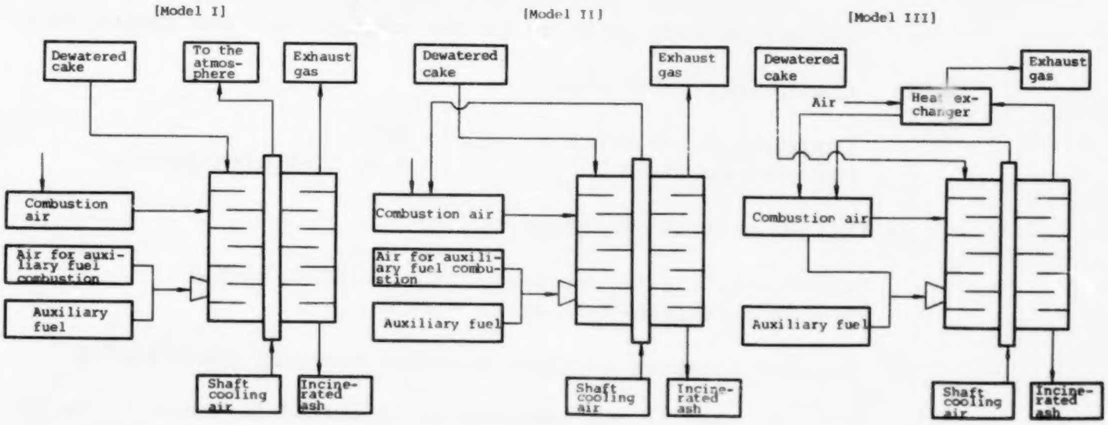

[Model I]
Dewatered cake
To the atmosphere
Exhaust gas
Combustion air
Air for auxiliary fuel combustion
Auxiliary fuel
Shaft cooling air
Incinerated ash
[Model II]
Dewatered cake
Exhaust gas
Combustion air
Air for auxiliary fuel combustion
Auxiliary fuel
Shaft cooling air
Incinerated ash
[Model III]
Dewatered cake
Air
Heat exchanger
Exhaust gas
Combustion air
Auxiliary fuel
Shaft cooling air
Incinerated ash

Fig. 11 Model incinerators

In Model I, the heat output A-2 was included in the balance. In Model II, A-2 became zero because it was recirculated as A-3. In Model III, A-2 was zero, too, and A-4 was added to the balance as a heat input in proportion to the heat output G-1.

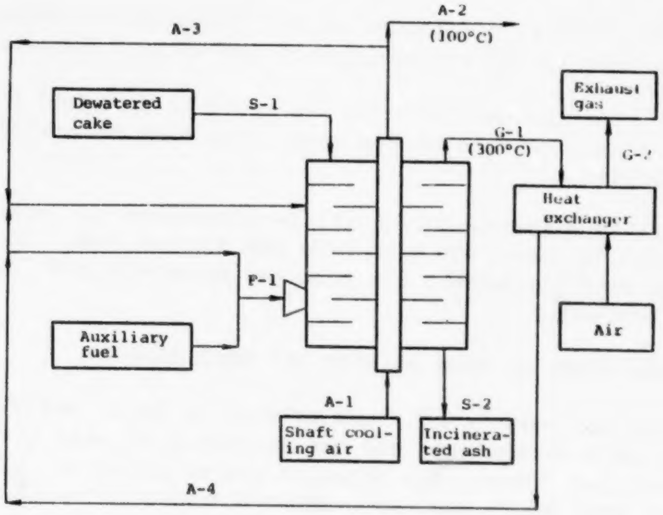

A-3
A-2
(100°C)
Dewatered cake
S-1
Exhaust gas
G-1
(300°C)
G-2
Heat exchanger
F-1
Auxiliary fuel
Air
A-1
S-2
Shaft cooling air
Incinerated ash
A-4

Fig. 12 Material flow of the model incinerator

(2) Basic numerical values used to compute the mass and heat balances

(a) Standard level of heat balance

Atmospheric temperature, and high-level standard (Heat associated with moisture in the supplied air was neglected.)

(b) Atmospheric conditions

Atmospheric temperature	T_o :	20°C
Absolute moisture	h :	0.01 kg-H_2O/kg-dry air

(c) Latent heat of vaporization

Latent heat at temperature T_o°C: L (Kcal/kg-H_2O)
L = 586 (Kcal/kg-H_2O) at T_o = 20°C

(d) Average constant pressure specific heat; C_p (Kcal/kg-°C)

C_p used in the calculation is listed in Table 8. As an average constant pressure specific heat, the value at the temperature (0 + T)/2°C was used. Also, as the dry combustion gas specific heat, the specific heat of nitrogen was used.

Table 8 The values of constant pressure specific heat used for the computation

Temperature T (°C)	Dry combustion gas C_{PG}	Vapor C_{PH}	Dry air C_{PA}	Range of applicable temperature
0 - 100	0.248	0.447	0.240	$T < 150$
0 - 300	0.250	0.457	0.243	$150 \leqq T < 400$
0 - 500	0.253	0.470	0.247	$400 \leqq T$

- o The constant pressure specific heat at $\{(0 + T)/2\}$°C is used as an average constant heat specific heat.
- o The specific heat nitrogen is used as the specific heat of dry combustion gas.

(e) Composition of dry air (weight base, and volume base)

O_2; 23.3 weight %, 21.0 vol %
N_2; 76.7 weight %, 79.0 vol %
Specific gravity; 1.29 (kg/N-m^3)

(f) Specific heat of ash (C_A)

$C_A = 0.3$ (Kcal/kg-°C)

(3) The general properties and basic numerical value relating to combustion of sludge and auxiliary fuel

(a) Properties: Values shown in Table 9 were used.

Table 9 Properties of cake and auxiliary fuel

Item		Cake		Auxiliary fuel Heavy oil A	
Specific gravity		S_G		F_G	0.84
Water content		S_W		F_W	0
Ignition loss		S_V		F_V	0
Higher heating value (Kcal/Kg-DS)		S_{HCV}	5,500	F_{HCV}	10,800
Inflammable constituents	C (%)	S_C	52	F_C	85.4
	H (%)	S_H	7	F_H	12.6
	N (%)	S_N	7	F_N	0.4
	S (%)	S_S	1.5	F_S	0.9
	O (%)	S_O	32.5	F_O	0.7

(b) Basic numerical values relating to combustion:

The basic numerical values and the equations to calculate the values are summarized in Table 10.

Table 10 Basic numerical values relating to combustion of cake and fuel

	Cake		Auxiliary fuel	
	Weight of gas (kg/kg-VS)	Volume of gas (Nm^3/kg-VS)	Weight of gas (kg/kg-VS)	Volume of gas (Nm^3/kg-VS)
Theoretical dry air	S_{TA} 7.03	S_{VA} 5.45	F_{TA} 14.10	F_{VA} 10.96
	$S_{TA} = (S_C \times 32/12 + S_H \times 16/2 + S_S \times 32/32 = S_O)/23.3$	$S_{VA} = (S_C \times 22.4/12 + S_H \times 11.2/2 + S_S \times 22.4/32 - S_O \times 22.4/32)/21$	$F_{TA} = (F_C \times 32/12 + F_H \times 16/2 + F_S \times 32/32 - F_O)/23.3$	$F_{VA} = (F_C \times 22.4/12 + F_H \times 11.2/2 + F_S \times 22.4/32 - F_O \times 22.4/32)/21$
Theoretical dry gas in the exhaust gas	S_{TD} 7.4	S_{VW} 5.34	F_{TD} 13.97	F_{VD} 10.26
	$S_{TD} = 1 + S_{TA} - S_{TW}$	$S_{VD} = S_C/100 \times 22.4/12 + S_N/100 \times 22.4/28 + S_S/100 \times 22.4/32 + S_{VA} \times 79/100$	$F_{TD} = 1 + F_{TA} - F_{TW}$	$F_{VD} = F_C/100 \times 22.4/12 + F_N/100 \times 22.4/28 + F_S/100 \times 22.4/32 + F_{VA} \times 79/100$
Water generated on burning	S_{TW} 0.63	S_{VD} 0.78	F_{TW} 1.13	F_{VW} 1.41
	$S_{TW} = S_H/100 \times 18/2$	$S_{VW} = S_H/100 \times 22.4/2$	$F_{TW} = F_H/100 \times 18/2$	$F_{VW} = F_H/100 \times 22.4/2$

(4) Operating conditions: The operating conditions of the Models were assumed as follows taking into account the operating conditions of actual furnaces.

S; Cake input into the furnace (kg-wet cake/hour)

ϕ_S; Ratio of actual air supply to the theoretical air requirement to burn inflammable constituents completely (theoretical requirement plus excess air, $1 + \alpha$)

ϕ_M; Air ratio relating to burning the auxiliary fuel

ϕ; Overall air ratio, combining the ratios ϕ_S and ϕ_M

The output of heat taken out by the shaft cooling air was assumed to be 2% of the total heat input:

T_{G1}; The temperature of the exhaust gas at the outlet of the furnace, which was assumed to be 300°C.

T_{S1}; The temperature of the incinerated ash, which was assumed to be 300°C.

η_L; The overall heat loss of the incinerator, which was set at 18% of the total heat input. ($\eta_L = \eta_{L1} + \eta_{L2}$)

η_{L1}; The heat loss (unburned loss) resulting from the unburned inflammable material, which was set as being 12% of the total heat input.

η_{L2}; The heat loss (radiation loss) resulting from the radiation from the incinerator itself, which was set as being 6% of the total heat input.

ε; The recovery efficiency of the heat in the exhaust gas by the heat exchanger, which was set as being 8% of the sensible heat of the exhaust gas at the furnace gas outlet.

(5) The materials balance at the multiple-hearth furnace

The quantity and quality of sludge input into the furnace are defined as follows:

S; Cake input into the furnace (kg-wet cake/hour)

S_{WO}; Water input into the furnace (kg/hour)

S_{DS}; Solid input into the furnace (kg/hour)

S_{VO}; Inflammable constituent input into the furnace (kg/hour)

S_{AS}; Mineral input into the furnace (kg/hour)

These parameters can be related by the following equations 12 to 15.

$S_{WO} = S \times S_W/100$ .... Equation 12

$S_{DS} = S - S_{WO}$ .... Equation 13

$S_{VO} = S_{DS} \times S_V/100$ .... Equation 14

$S_{AS} = S_{DS} - S_{VO}$ .... Equation 15

Also, when the auxiliary fuel supply and the combustion air supply are define as F (kg/hour) and A (kg/hour), respectively, the dry gas flow D_{G1} and water output H_{G1} (kg/hour) in the exhaust gas at the furnace outlet can be expressed by Equations 16 and 17.

$D_{G1} = S_{VO} \{S_{TD} + (\phi - 1)\} S_{TA} + F \{F_{TD} + (\phi - 1)\} F_{TA}$
$= S_{VO} \{7.4 + (\phi - 1)\ 7.03\} + F \{13.97 + /$
$(\phi - 1)\ 14.1\}$ .... Equation 16

$$H_{G1} = S_{VO}\{S_{TW} + \phi \cdot S_{TA} \cdot h\} + F\{F_{TW} + \phi \cdot F_{TA} \cdot h\} + S_{WO}$$
$$= S_{VO}\{0.63 + o \times 7.03 \times 0.01\} + F\{1.13 + \phi \times 14.1 \times 0.01\} + S_{WO} \quad \text{.................................. Equation 17}$$

(6) The heat balance at the multiple hearth furnace

From the material balance described above, the input and output heat of a furnace can be expressed by Equations 18 to 27.

Heat Input

The heat that sludge generates (Q_1) (Kcal/hour)

$$Q_1 = S_{VO} \times S_{HCV} = S_{VO} \times 5{,}500 \quad \text{.......................... Equation 18}$$

The heat that auxiliary fuel generates (Q_2) (Kcal/hour)

$$Q_2 = F \times F_{HCV} = F \times 10{,}800 \quad \text{............................ Equation 19}$$

The sensible heat of the supplied air (Q_3) (Kcal/hour)

$$Q_3 = Q_4 \times \varepsilon/100 = Q_4 \times 0.08 \quad \text{........................... Equation 20}$$

(Q_3 is added to the heat input only for Model III.)

Heat Output

Sensible heat of the gas at the furnace gas outlet (Q_4) (Kcal/hour)

$$Q_4 = (D_{G1} \cdot C_{PG} + H_{G1} \cdot C_{PH}) \times (T_{G1} - T_0)$$
$$= (D_{G1} \times 0.25 + H_{G1} \times 0.457) \times (300-20) \quad \text{............ Equation 21}$$

Latent heat of vaporisation for water in the sludge (Q_5) (Kcal/hour)

$$Q_5 = S_{WO} \times L = S_{WO} \times 586 \quad \text{.............................. Equation 22}$$

Latent heat of heatable constituent in the sludge and of water generated by burning (Q_6) (Kcal/hour)

$$Q_6 = S_{VO} \times S_{TW} \times L = S_{VO} \times 0.63 \times 586 \quad \text{................. Equation 23}$$

Latent heat of auxiliary fuel and of water generated by burning (Q_7) (Kcal/hour)

$$Q_7 = F \times F_{TW} \times L = F \times 1.13 \times 586 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad \text{Equation 24}$$

Sensible heat of ash (Q_8) (Kcal/hour)

$$\begin{aligned} Q_8 &= S_{AS} \times C_A \times (T_{S1} - T_0) \\ &= S_{AS} \times 0.3 \times (300 - 20) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \text{Equation 25} \end{aligned}$$

Sensible heat of the shaft cooling air (Q_9) (Kcal/hour)

$$Q_9 = (Q_1 + Q_2) \times \eta/100 = (Q_1 + Q_2) \times 0.02 \quad \ldots\ldots\ldots\ldots \quad \text{Equation 26}$$

(Q_9 is added to the heat output only for Model I.)

Heat loss (Q_{10}) (Kcal/hour)

$$Q_{10} = (Q_1 + Q_2) \times \eta/100 = (Q_1 + Q_2) \times 0.18 \quad \ldots\ldots\ldots\ldots \quad \text{Equation 27}$$

Using Q_1 to Q_{10} mentioned above, a heat balance equation can be formulated by making [heat input] = [heat output].

$$Q_1 + Q_2 + Q_3 + Q_5 + Q_6 + Q_7 + Q_8 + Q_9 + Q_{10} \quad \ldots\ldots\ldots\ldots \quad \text{Equation 28}$$

The required amount of auxiliary fuel (F_n kg/hour) for each model can be expressed by the Equations 29 to 31.

The required amount of auxiliary fuel for Model I

$$\begin{aligned} F_A = \Big\{ & \frac{714 S_W}{100} + 84 \left(1 - \frac{S_W}{100}\right) \left(1 - \frac{S_V}{100}\right) - \left(1 - \frac{S_W}{100}\right) \\ & \times \frac{S_V}{100} \times (3{,}914 - 501\phi) \Big\} \times \frac{S}{7{,}840 - 1{,}006\phi} \quad \ldots\ldots\ldots\ldots \quad \text{Equation 29} \end{aligned}$$

The required amount of auxiliary fuel for Model II

$$\begin{aligned} F_B = \Big\{ & \frac{714 S_W}{100} + 84 \left(1 - \frac{S_W}{100}\right) \left(1 - \frac{S_V}{100}\right) - \left(1 - \frac{S_W}{100}\right) \\ & \times \frac{S_V}{100} \times (4{,}034 - 501\phi) \Big\} \times \frac{S}{8{,}056 - 1{,}006\phi} \quad \ldots\ldots\ldots\ldots \quad \text{Equation 30} \end{aligned}$$

The required amount of auxiliary fuel for Model III

$$\begin{aligned} F_C = \Big\{ & \frac{714 S_W}{100} + 84 \left(1 - \frac{S_W}{100}\right) \left(1 - \frac{S_V}{100}\right) - \left(1 - \frac{S_W}{100}\right) \\ & \times \frac{S_V}{100} \times (4{,}034 - 461\phi) \Big\} \times \frac{S}{8{,}067 - 925\phi} \quad \ldots\ldots\ldots\ldots \quad \text{Equation 31} \end{aligned}$$

4.3 Heat Characteristics Diagram of the Multiple-hearth Furnace

Based on the heat balance Equations of 29 to 31 for each Model, a characteristic diagram of the amount of auxiliary fuel requirement was calculated by taking the combustion air ratio (ϕ), ignition loss of the input cake (S_V), and the moisture content of the cake (S_W) as the parameters.

(1) Parameter setting

(a) The combustion air (ϕ) were set as being five conditions: 1.3, 1.6, 2.0, 2.5, and 3.0.

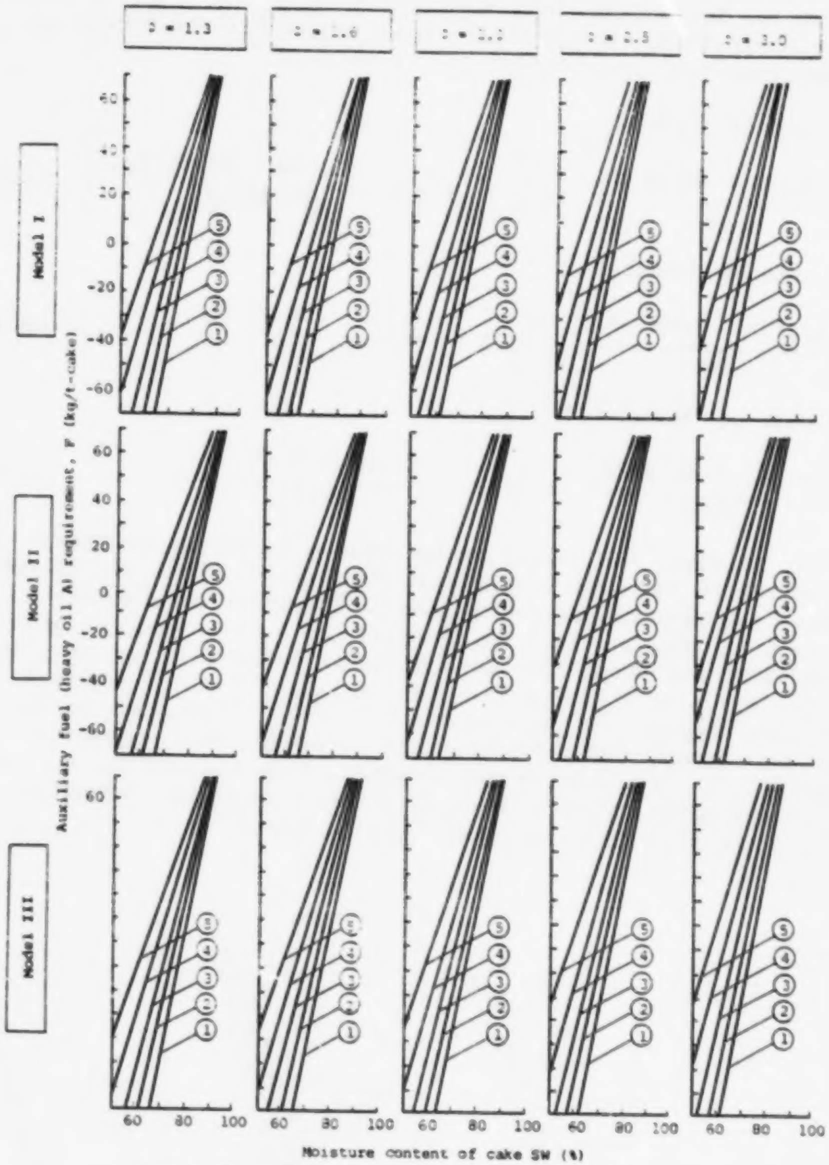
(b) In the computation of the heat balance, the heat properties of cake that need to be given a numerical value are the higher heating value of ignition loss (S_{HCV}), the overall ignition loss (S_V), and the hydrogen content in ignition loss. Here, the higher heating value of ignition loss has been assumed as being 5,500 (kcal/kg-VS), and hydrogen content in ignition loss being 7% (weight ratio). Therefore, the only variable factor is ignition loss (S_V). As shown in Table 11, the five levels of ignition loss were assumed, and the corresponding heating values were calculated.

Table 11 The relations between ignition loss and the higher heating value of cake

Classification No.	Ignition loss S_V	Ash content S_A	Higher heating value based on ignition loss S_{HCV} (Kcal/kg-VS)	Higher heating value based on cake solids $S_{DCV} = S_{HCV} \times S_V/100$ (Kcal/kg-DS)
①	80	20	5,500	4,400
②	70	30		3,850
③	60	40		3,300
④	50	50		2,750
⑤	40	60		2,200

(c) Five different levels of moisture content (S_W) were set: 60%, 65%, 70%, 75%, and 80%.

Fig. 13 shows the amount of auxiliary fuel (calculated in terms of heavy oil A) required to burn one ton of dewatered cake under the combinations of the moisture content (S_W) and ignition loss (S_V). In the diagram, when the amount of auxiliary fuel is shown to be negative, it indicates a status in which the cake can provide a sufficient heat, and thereby needs no auxiliary fuel.



Numerals ① to ⑤ in the diagrams indicate the conditions of calssified numbers in Table - 11.

Fig. 13 The relationship between the moisture content of cake and the amount of auxiliary fuel (heavy oil A) requirement for each model

4.4 Energy-saving Incineration

To clearly define the situation of autogeneous combustion, Equations 29, 30, and 31, were modified to give the relation between the moisture content (S_W) and ignition loss (S_V) under the condition that auxiliary fuel requirement (F) is zero.

In the case of Model I

$$S_W = \left\{\frac{(3{,}998 - 501\phi)\, S_V}{100} - 84\right\} / \left\{6.3 + \frac{(3{,}998 - 501\phi)\, S_V}{10{,}000}\right\} .. \quad \text{Equation 32}$$

In the case of Model II

$$S_W = \left\{\frac{(4{,}118 - 501\phi)\, S_V}{100}\right\} / \left\{6.3 + \frac{(4{,}118 - 501\phi)\, S_V}{10{,}000}\right\} .... \quad \text{Equation 33}$$

In the case of Model III

$$S_W = \left\{\frac{(4{,}118 - 461\phi)\, S_V}{100}\right\} / \left\{6.3 + \frac{(4{,}118 - 461\phi)\, S_V}{10{,}000}\right\} .... \quad \text{Equation 34}$$

Fig. 14 shows the relationship between S_W and S_V by using Equations 32 to 34. The line where the auxiliary fuel (F) becomes zero is a marginal line of autogenous combustion under this model. Therefore, in Fig. 14, the region above the curve shows the area where the condition for autogenous combustion is satisfied, and the region below the curve is the area in which auxiliary fuel is required. By the use of this diagram, it is possible to make an estimation on the conditions for S_W, S_V, and ϕ that are necessary for autogenous combustion. It should be noted, however, that besides S_W, S_V, and ϕ, there are other factors that may affect autogenous combustion, including the type of furnace and burning speed.

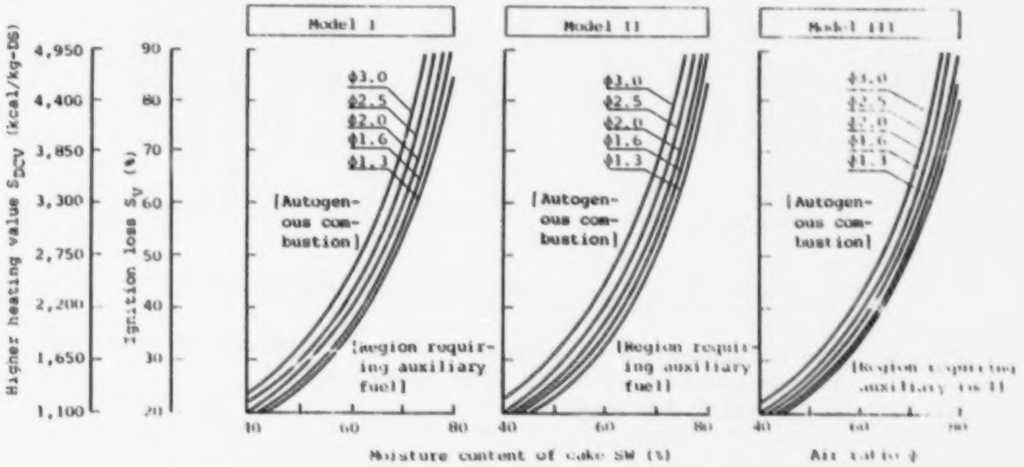


Fig. 14 Limit of autogenous combustion for each model

4.5 Evaluation of the Existing Multiple Hearth Furnaces by the Theoretical Model

There have been four units of multiple-hearth furnaces in service, about which enough data are available for being evaluated with the theoretical model. Of these four furnaces, two are operated without auxiliary fuel, and the other two need auxiliary fuel. Because existing available data were used, some of them might not fully be satisfactory in accuracy.

Table 12 shows the actual operating conditions of these four multiple-hearth furnaces. The data of treatment plant A are considered to have high accuracy. The furnace of this plant has been incinerating sludge without auxiliary fuel. The data of treatment plant B were obtained during the test run of the furnace, and therefore, only a single set of data was available as to element compositions and heating values. The furnace, however, was maintained at the state of autogenous combustion. The data of treatment plant C were obtained as the results of investigations to grasp the heat balance of the furnace, and are considered to have a higher accuracy. The furnace was using some auxiliary fuel. The data treatment plant D were obtained during a test run. The heat balance calculated directly from these data showed that the heat output was greater than the input. The data was not so satisfactory from the viewpoint of accuracy.

Table 12 The results of survey on the actual operating conditions of the multiple-hearth furnaces

Name of sewage treatment plant		Sewage treatment plant A (30 ton/day furnace)							Sewage treatment plant B (150 ton/day furnace)					Sewage treatment plant C (200 ton/day furnace)				Sewage treatment plant D (150 ton/day furnace)
Run		1	2	3	4	5	6	B-1	B-2	B-3	4	5	6	1	2	3	4	1
Rate of cake feed (kg/hour)		1,692	1.429	1.521	1.088	1.021	1.038	4.030	3.110	3.230	4,960	4,590	4,660	4,638	4,163	8,333	8,410	6,200
Loading ratio (%)		135	114	122	87	82	83	64.5	49.8	51.7	79.4	73.4	74.6	55.7	50.0	100	101	100
Dewatered cake	Classification (%)	Polymer, Belt-press	*	*	*	*	*	Ferric chloride and lime, Filter press	*	*	*	*	*	Polymer, Centrifuge	*	*	*	Polymer and centrifuge: 2 Lime and vacuum filter: 1
	Moisture conent (%)	72.2	74.9	75.7	74.7	77.2	83.6	58.5	64.1	61.5	65.0	61.5	64.7	78.2	81.0	78.0	75.2	80
	Ignition loss (%)	70.5	69.9	69.2	68.8	68.2	69.8	47.2	*	*	*	*	*	70.2	68.0	66.7	61.6	45
	Element composition: Carbon (%)	53.9	53.4	52.2	52.1	52.6	54.6	48.40	*	*	*	*	*	52.0	52.0	52.0	52.0	50.9
	Element composition: Hydrogen (%)	7.7	6.6	6.8	7.0	7.0	6.4	8.25	*	*	*	*	*	8.0	8.0	8.0	8.0	6.9
	Element composition: Nitrogen (%)	7.7	7.6	7.1	6.7	6.7	9.9	7.17	*	*	*	*	*	6.0	6.0	6.0	6.0	5.0
	Element composition: Sulphur (%)	0.9	0.9	0.9	0.9	0.9	0.9	0.38	*	*	*	*	*	1.5	1.5	1.5	1.5	0.3
	Element composition: Oxygen (%)	29.8	31.5	33.0	33.3	32.9	28.6	35.80	*	*	*	*	*	32.5	32.5	32.5	32.5	35.9
	Higher heating value per ignition loss (kcal/kg-VS)	5,730	5,708	5,621	5,654	5,689	5,873	5,617	*	*	*	*	*	5,670	5,284	5,039	4,934	4,667
	Higher heating value per dry solids (kcal/kg-DS)	4,040	3,990	3,890	3,890	3,880	4,100	2,651	*	*	*	*	*	3,980	4,280	3,930	3,710	2,100
	Lower heating value per wet cake (kcal/kg-cake)	608	490	429	470	363	131	662	492	571	464	571	474	332	271	333	403	-93.6
Exhaust gas at the furnace outlet	Temperature (°C)	450	398	362	359	284	264	418	298	294	318	381	270	280	290	340	372	330
	Dry gas flow (Nm³/hour)	5,610	3,871	3,701	3,359	2,889	3,830	13,131	9,798	11,471	12,508	13,484	16,964	6,690	8,280	11,310	13,520	9,700
	Wet gas flow (Nm³/hour)	4,017	2,714	2,365	2,371	1,916	2,627	3,891	3,140	3,213	4,991	4,520	4,763	12,341	15,271	22,780	25,744	16,000
Air ratio (-)		1.89	1.30	1.45	1.78	1.36	Unclear	3.12	3.49	3.67	2.86	3.03	4.11	1.34	2.16	1.45	1.82	1.68
Auxiliary fuel consumption (kg/hour)		0	0	0	0	0	Heavy oil A 25.9	0	0	0	0	0	0	Kerosene 79.8	119.3	132.7	101.9	Heavy oil A 986 (kg/hour), Digestion gas 184 kg/hour
Unburned loss in the ash (dry %)		0.66	0.15	0.5	0.33	0.1	1.24	Unclear	Unclear	Unclear	Unclear	Unclear	Unclear	6.98	6.26	6.87	6.67	Unclear
Remarks														[illegible]	117.4(kg/hour) 20.2(kg heavy oil/ton)	135.2(kg/hour) 16.2(kg heavy oil/ton)	103.8(kg/hour) 12.3(kg heavy oil/ton)	Converted to heavy oil A; 194 (kg/hour), 31.3 kg heavy oil per ton of cake

Because all four furnaces are the Model II type, the data are plotted on the characteristic diagram of Model II of Fig. 11 putting the air ratio ϕ equal to 1.6, which is shown in Fig. 15. Similarly, data were plotted on the diagram of Fig. 12, and the result is shown in Fig. 16.

The Run (A-1) in Fig. 15 corresponds to Run 1 of treatment plant A. As shown in Table 12, the moisture content of the Run 1 is 72.2%, and the higher heating value is 4,040 (Kcal/kg), and the consumption of auxiliary fuel is zero. On the other hand, when the auxiliary fuel requirement is sought on Fig. 15 using the diagram with the higher heating value of 3,850 Kcal/kg-DS, it is found to be -17 kg/t-cake, which means that incineration is not only autogenous combustion but also energy producing.

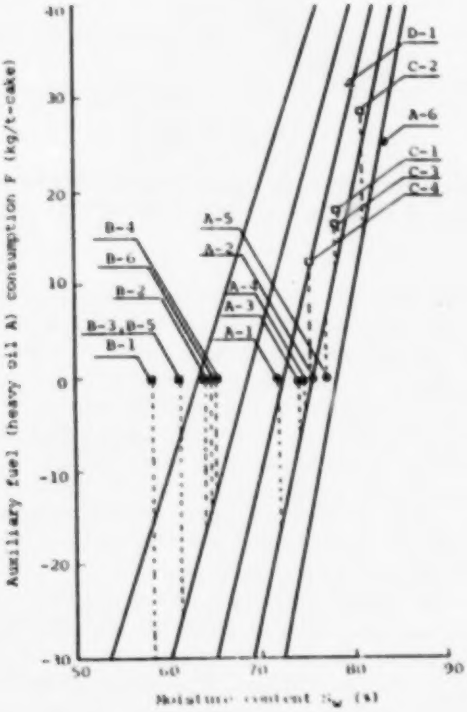


Fig. 15 Comparison between the fuel requirement calculated on Model II at ϕ = 1.6 and actual value

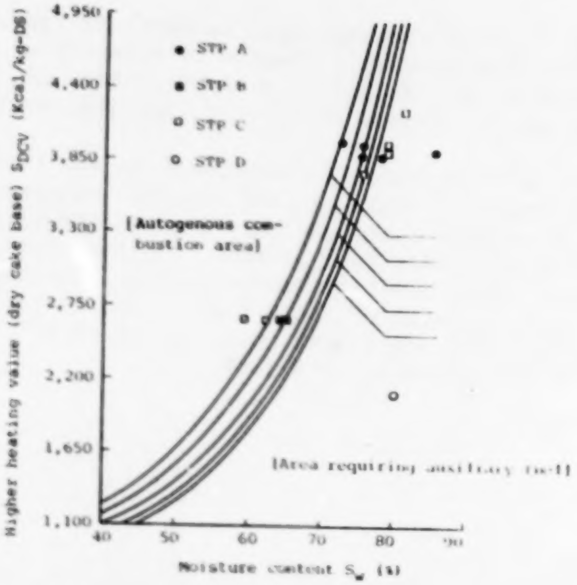


Fig. 16 Comparison between the theoretical boundary for autogenous combustion for Model II and the actual state

In the case of Run (C-1), auxiliary fuel type was being used. With moisture content of 78.2 and the higher heating value of 3,980 (Kcal/kg-DS), the amount of auxiliary fuel consumption was 17.5 (kg/t-cake). On the other hand, the auxiliary fuel consumption estimated from Fig. 13 under these condition is 11 (kg/t-cake) of heavy oil A using the values of $\phi = 1.6$ and the higher heating value of 3,850 (Kcal/kg-DS), thereby indicating the possibility of saving more fuel.

In the both figures (Figs. 15 and 16), data from the sewage treatment plants A and B that are performing the autogenous combustion, fell on the autogenous combustion region with a few exceptions, showing the efficient operation of the plants.

Especially, the dewatered cake of treatment plant B has a greater reserve in energy, exceeding the lime of autogenous combustion, and is of substantial value as fuel. At treatment plant B, the air ratio is being raised to 3.1 - 4.1 to meet the large heat input by making the sensible heat of exhaust gas greater. A furnace of this kind, however, can produce energy by recovering surplus heat to use it for other purposes.

The dewatered cake of treatment plant A has a characteristics around the limit of autogenous combustion. It should be noted that although the calculation on Run A-5 shows a fuel requirement of 6 (kg/t-cake) of heavy oil, in reality, the furnace remained in a state of autogenous combustion. These data were obtained during experiments to clarify the critical moisture content for autogenous combustion, and the operation of the furnace might not be so stable. Nevertheless, the furnace has a high burning efficiency. The air ratio ϕ at that time was 1.36, and the exhaust gas temperature was 284°C. Therefore, the exhaust gas sensible heat output was small.

Both furnaces of treatment plants C and D are operated with the auxiliary fuel addition. Plant C was found to use a slightly larger amount of auxiliary fuel than that expected by the calculation on the Model II. This was because the total heat loss including unburned loss, shaft-cooling loss, and radiation loss was considerably large with the maximum loss of 25%, which was much greater than the heat loss of 18% used in the computation. In addition, the exhaust gas temperature, 340°C and 372°C for Run C-3 and C-4, respectively, was higher than 300°C assumed in the computation, resulting in a greater heat output.

The heat balance computed by using treatment D's data reveal that the input was 4,680,000 (Kcal/hour), whereas the amount of heat output was 5,180,000 (Kcal/hour), indicating a problem in the reliability of data.

Based on these studies using the models and observations on the actual statuses, the possibility of energy saving in incineration by focusing on the input and output of heat can be summarized as follows.

(a) The heating value of the cake: It is important to make the heating value of the cake as high as possible which means to decrease the moisture content and to increase the organic content. An effort should be made to use the dehydrating system capable of producing such cake. Fig. 13 could be a reference to determine the target property of the cake.

(b) Exhaust gas sensible heat: The exhaust gas sensible heat of a multiple-sheath furnace is normally 20% to 40% of the heat input. To reduce the sensible heat, it is important that the exhaust gas outlet temperature be lowered. If the heat exchange efficiency increase during sludge drying at the upper part of the furnace, the exhaust gas temperature can be lowered. Another effective method is to lower the wet gas sensible heat in the exhaust gas. Decrease in the air ratio to minimize the amount of exhaust gas may greatly affect the reduction of the sensible heat.

(c) Sensible heat of water: The water in the exhaust gas consists of water contained in the cake and water produced during the incineration by the reaction of hydrogen in organics with oxygen. Although the sensible heat of water vary greatly depending on the moisture content of the cake, it is usually equivalent to 40% - 70% of the heat input. Because the heat output due to sensible heat of water cannot be controlled by manipulating the furnace, attaining a low moisture content by the dehydrating process is necessary.

(d) Sensible heat of incinerated ash: The sensible heat of ash accounts for only about 0.1% of the total sensible heat loss. Therefore, there is very little possibility of recovering a substantial amount of energy from the ash that may greatly affect total energy recovery.

(e) Sensible heat of the shaft cooling air: It is difficult to control this sensible heat because the air flow is fixed at the time of design, and the temperature at outlet is also fixed at a constant level. The heat recovery, however, is possible by circulating the heated air into the furnace. The sensible heat of the shaft cooling air is normally equivalent to 1% - 5% of the total heat input.

(f) Heat loss: The heat loss mainly consists of the radiation heat loss from the furnace wall and the heat loss due to unburned combustible material. The former can be reduced by enhancing the heat-insulating efficiency of the furnace's wall structure. The heat radiation is equivalent to about 2% - 7% of the total heat input. The heat loss due to the unburned combustible materials is unavoidable to some extent when incinerating such a highly inflammable material with high volatile solids. Therefore, a technique capable of burning the heated wind containing unburned fuel to recover energy at the upper hearth of the furnace may become necessary. The amount of unburned material depends on the property of cake (content of volatile solids with low boiling point and so forth), and the heat loss ranges from 1% to 20% of the total heat input.

5. EVALUATION OF THE FLUIDIZED BED FURNACE FOR ENERGY SAVING

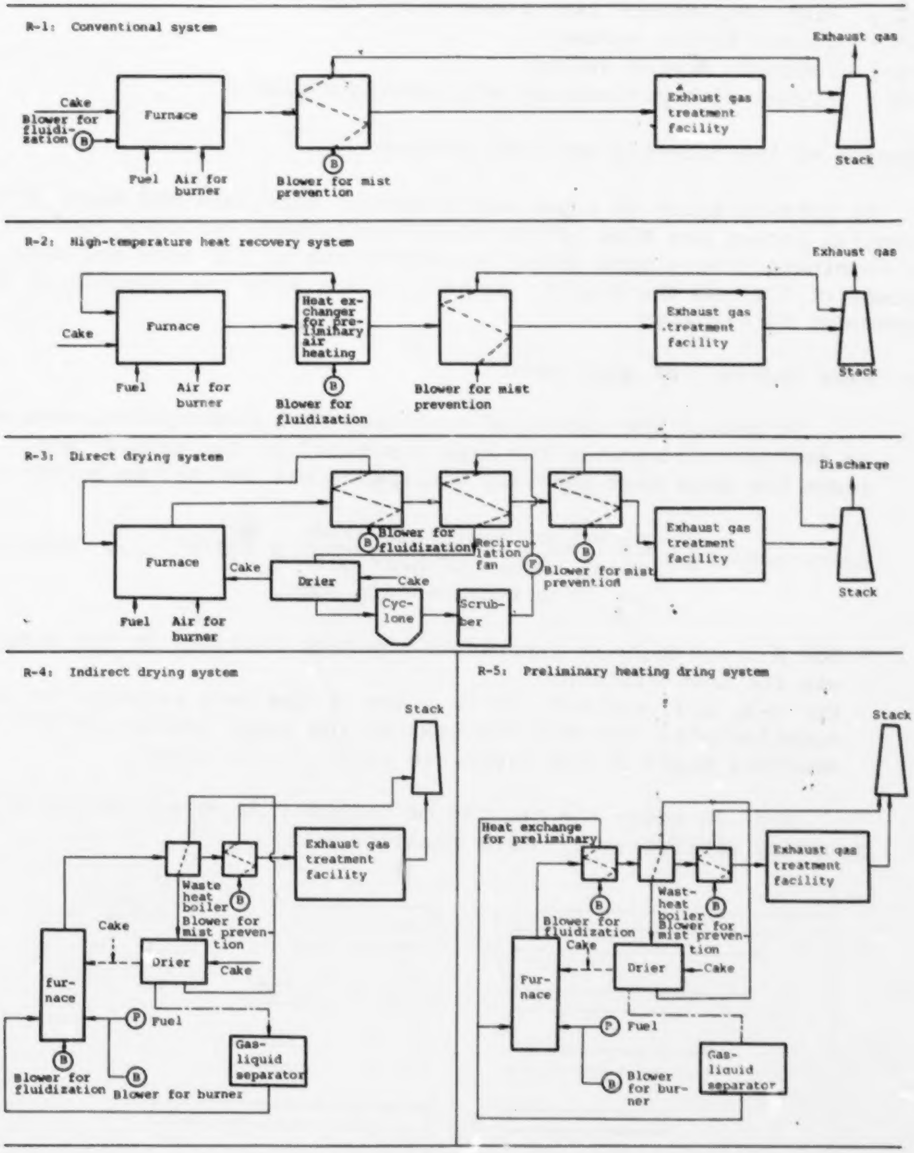

R-1: Conventional system
Exhaust gas
Cake
Blower for fluidization
Furnace
Fuel
Air for burner
Blower for mist prevention
Exhaust gas treatment facility
Stack
R-2: High-temperature heat recovery system
Exhaust gas
Cake
Furnace
Heat exchanger for preliminary air heating
Fuel
Air for burner
Blower for fluidization
Blower for mist prevention
Exhaust gas treatment facility
Stack
R-3: Direct drying system
Discharge
Furnace
Cake
Drier
Cake
Blower for fluidization
Recirculation fan
Blower for mist prevention
Cyclone
Scrubber
Exhaust gas treatment facility
Stack
Fuel
Air for burner
R-4: Indirect drying system
Stack
Exhaust gas treatment facility
Waste heat boiler
Blower for mist prevention
Cake
Drier
Cake
furnace
Fuel
Gas-liquid separator
Blower for fluidization
Blower for burner
R-5: Preliminary heating dring system
Stack
Heat exchange for preliminary
Blower for fluidization
Wast-heat boiler
Blower for mist prevention
Exhaust gas treatment facility
Cake
Drier
Cake
Furnace
Fuel
Blower for burner
Gas-liquid separator

Fig. 17 Flow chart of each model of the fluidized bed furnace

For the five models shown in Fig. 17, equations with respect to material and heat balances were formulated, and the methods for saving energy were reviewed in the same manner as described in Chapter 4 for the multiple-hearth furnace.

The models illustrated in Fig. 17 are:

R-1 ; Conventional system
R-2 ; High-temperature heat recovery system
R-3 ; Direct drying system
R-4 ; Indirect drying system
R-5 ; Drying with preliminary air heating system

5.1 Features of the Material and Heat Balance

In consideration of input and output of materials and heat, the essential parameters that affect energy-efficiency are described below. The examination were made under the conditions of the cake moisture content of 80% and the organic content of 70% with the capacity of the furnace of 50 (ton/day).

(1) Heat recovery of each model

To reduce the amount of auxiliary fuel consumption, each model is designed to recover the heat contained in the exhaust gas. An index for this heat recovery efficiency was defined as being:

$$\text{Heat-recovery} = \frac{\text{Amount of heat recovered*}}{\text{Total amount of heat output from furnace main body}} \times 100(\%) \quad \ldots \quad \text{Equation 37}$$

* For R-1 and R-2, it represents the heat retained in the supplied air for incineration.
For R-3, R-4, and R-5, it is a sum of the heat retained in the supplied air, the sensible heat of the cake, and the latent and sensible heats of the vaporized water in the dryer.

Fig. 18 shows the results of computation on each model's heat recovery efficiency by using Equation 37.

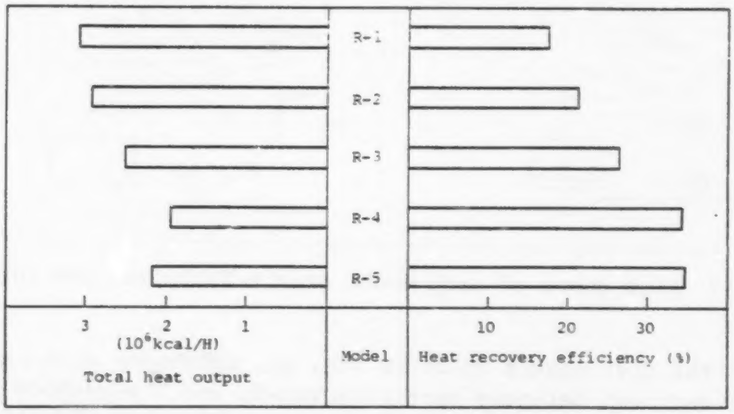


Fig. 18 Heat recovery efficiency of each model

As seen from this diagram, models with a higher heat recovery efficiency are those provided with an indirect dryer. In both R-4 and R-5, their heat recovery efficiency was the same, 32%, followed by 27% of R-3, 22% of R-2, and 18% of R-1.

The total heat output of each model is also shown in Fig. 18, and its ratio, based on R-1, is shown in Table 13-(1).

The R-4 has the least amount of heat output followed by the R-5, R-3, R-2, and R-1. The total heat output of R-4 is substantially reduced to 61% of that of R-1.

(2) Auxiliary fuel consumption

The auxiliary fuel consumption of each model is shown in Fig. 19. Model R-4 has the least auxiliary fuel consumption, about 16 (kg/H), followed by 19 (kg/H) of R-5, 36 (kg/H) of R-3, 63 (kg/H) of R-2, and 83 (kg/H) of R-1. The ratio of the fuel consumption of each model based on R-1 is shown in Table 13-(2).

Table 13-(1) Comparison of the total heat output of each model based on R-1

Model	R-1	R-2	R-3	R-4	R-5
$\frac{\text{Total heat output}}{\text{Total heat output of R-1}} \times 100$	100.0	96.4	81.1	61.1	72.1

Table 13-(2) Ratio of the auxiliary fuel requirement of each model based on R-1

Model	R-1	R-2	R-3	R-4	R-5
$\frac{\text{Auxiliary fuel requirement of each model}}{\text{Auxiliary fuel requirement of R-1}} \times 100$	100.0	75.9	43.4	19.3	22.9

The table clearly indicates that R-4 is capable of saving an amount of fuel consumption as high as 81% of the fuel consumption of R-1.

(3) Exhaust gas

The exhaust gas can be divided into dry gas and water. The quantity of dry gas varies depending on the amount of auxiliary fuel consumption when the air ratio, and the property and quantity of the cake input are the same. Therefore, the less the auxiliary fuel consumption, the smaller the amount of dry gas. The greater part of the water in the cake.

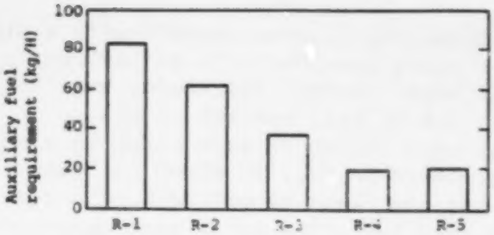


Fig. 19 Auxiliary fuel requirement of each model

The amounts of dry gas and water in the exhaust gas of each model are shown in Fig. 20.

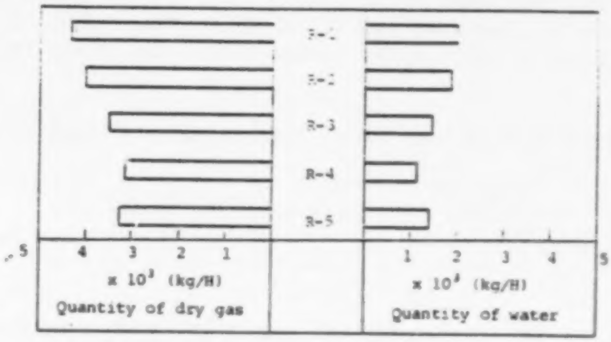


Fig. 20 Quantities of dry gas and water in the exhaust gas

The ratio of the dry gas and water in the exhaust gas of each model is shown in Table 14.

Table 14 Comparisons of the quantities of dry gas and water in exhaust gas, and the cake moisture content

Model	R-1	R-2	R-3	R-4	R-5
$\frac{\text{Dry gas of each model}}{\text{Dry gas of R-1}} \times 100$	100.0	91.4	80.4	72.1	73.0
$\frac{\text{Water in exhaust gas of each model}}{\text{Water in exhaust gas of R-1}} \times 100$	100.0	98.6	81.3	55.4	71.4
$\frac{\text{Cake moisture content of each model}}{\text{Cake moisture content of R-1}} \times 100$	100.0	100.0	81.5	52.7	71.2

In R-1 and R-2, the quantities of dry gas and water in the exhaust gas were determined by the auxiliary fuel consumption because the cake is put into the furnace without being dried. Therefore, the improvement made by R-2 is not so significant, a reduction of 19% of the quantity of dry gas and a reduction of only 1% of the water content. Conversely, in R-3, R-4, and R-5, the cakes were put into the furnaces after drying process.

The moisture contents of the cakes supplied to the furnaces were reduced from 80% to 76.5% in R-3, to 67.5% in R-4, and to 74.0 in R-5. As shown in Fig. 21, the amount of water brought into the furnaces also decreased. The amount of water brought into the furnace in R-4 was smaller by 47% than that in R-1.

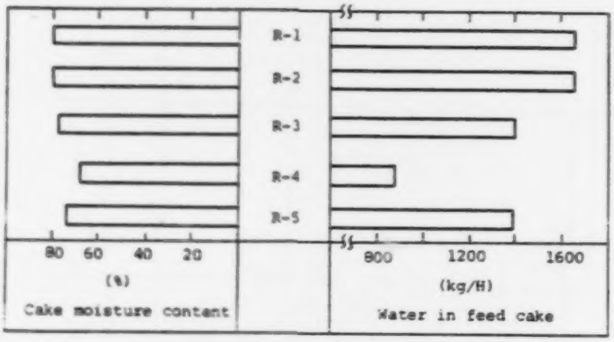


Fig. 21 Water in feed cake and cake moisture content

Consequently, the latent heat and sensible heat corresponding to the amount of water evaporated by drying process becomes unnecessary, thereby greatly reducing the auxiliary fuel consumption. This also reduces the quantity of dry gas and water in the exhaust gas. Table 14 shows the ratios of reduction in the amounts of dry gas and water in the R-3, R-4, and R-5. Model R-4 showed the highest reduction.

(4) Cake volume

The changes in the moisture content of the cake and water brought into the furnace by means of drying are shown in Fig. 21. Because R-3, R-4, and R-5 perform the drying, the volume of cake fed into the furnaces decreased. The ratios of reduction of cake volume based on R-1 are shown in Table 15. The greatest reduction was attained by R-4 which was 62% of R-1.

The changes in the heating values based on wet cake are shown in Table 15. The cake of R-4 showed the highest usable heating value of 762 (Kcal/kg-cake). The heating value of the cake of R-4 was about three times greater than that of R-1.

As typical examples of the material and heat balances, the case of Run-1 is shown in Fig. 22 and that of Run-2 in Fig. 23.

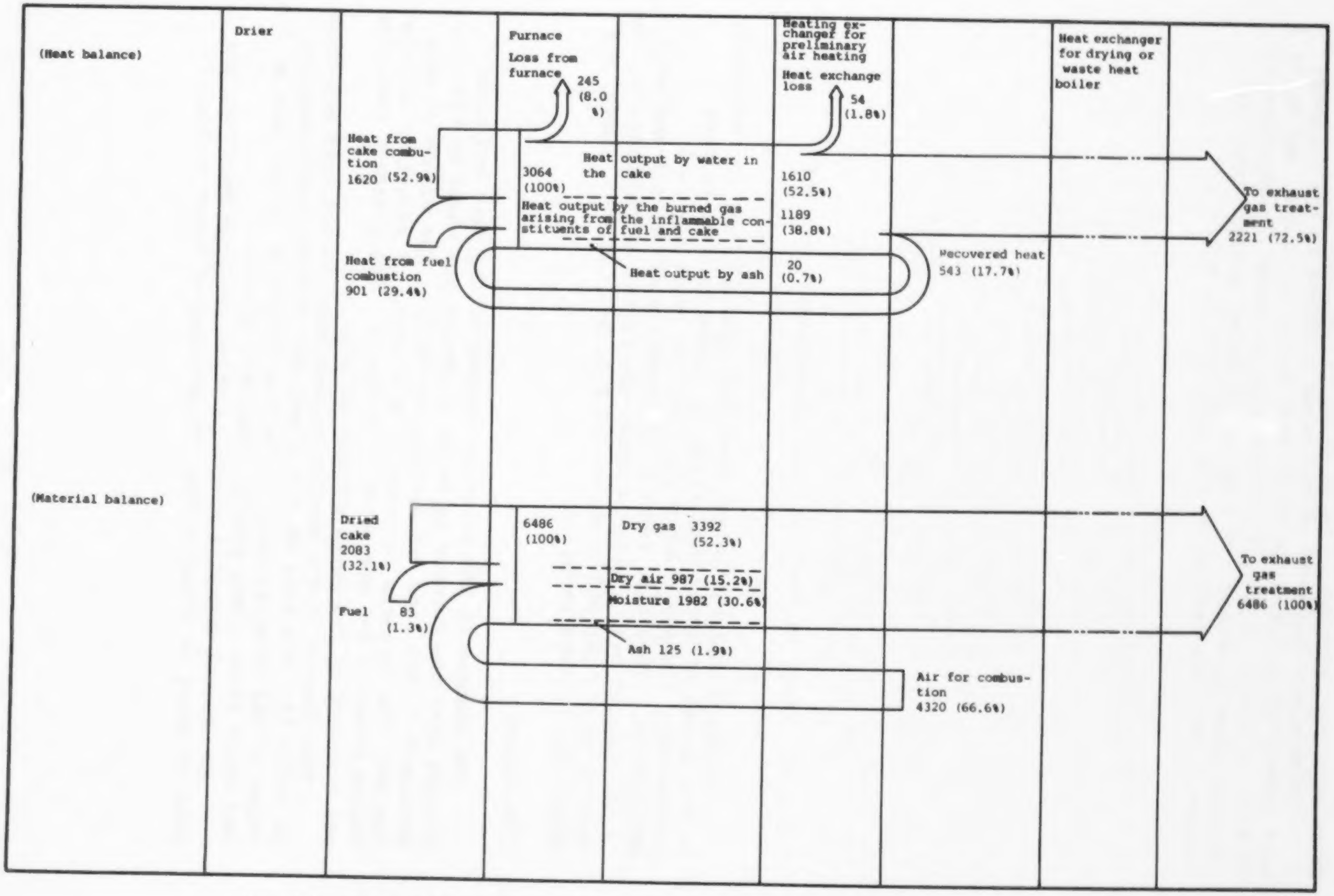

(Heat balance)
Drier
Furnace
Heating ex-changer for preliminary air heating
Heat exchanger for drying or waste heat boiler
Loss from furnace
245 (8.0 %)
Heat exchange loss
54 (1.8%)
Heat from cake combu-tion 1620 (52.9%)
3064 (100%)
Heat output by water in the cake
1610 (52.5%)
Heat output by the burned gas arising from the inflammable con-stituents of fuel and cake
1189 (38.8%)
To exhaust gas treat-ment 2221 (72.5%)
Heat from fuel combustion 901 (29.4%)
Heat output by ash
20 (0.7%)
Recovered heat 543 (17.7%)
(Material balance)
Dried cake 2083 (32.1%)
6486 (100%)
Dry gas 3392 (52.3%)
Dry air 987 (15.2%)
Moisture 1982 (30.6%)
Fuel 83 (1.3%)
Ash 125 (1.9%)
To exhaust gas treatment 6486 (100%)
Air for combus-tion 4320 (66.6%)

Fig. 22 Material and heat balance of R-1 (Cake moisture content of 80% and organic content of 70%; incinerator capacity of 50 ton/day)

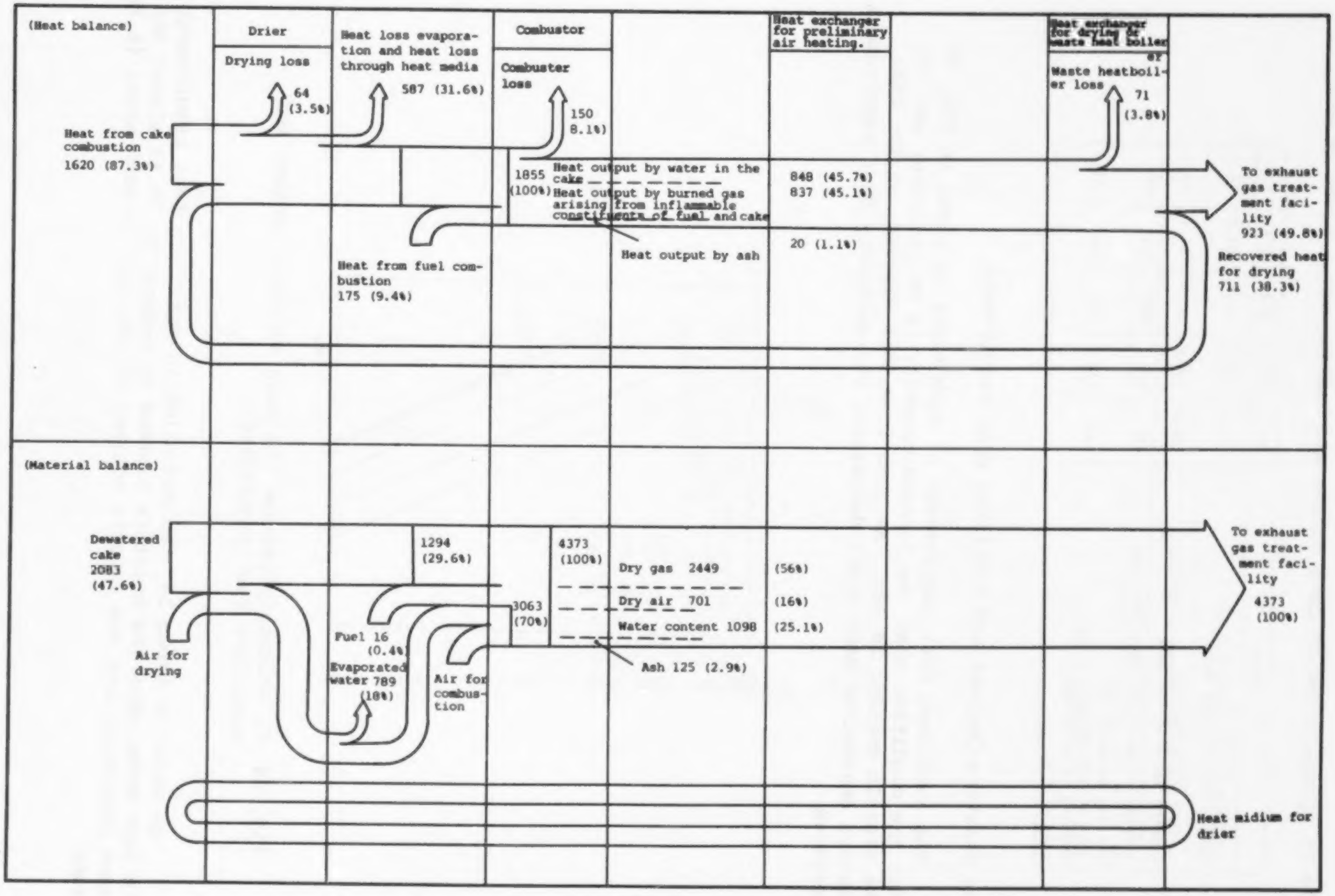

(Heat balance)
Drier
Heat loss evaporation and heat loss through heat media
Combustor
Heat exchanger for preliminary air heating.
Heat exchanger for drying or waste heat boiler
Drying loss
64 (3.5%)
587 (31.6%)
Combuster loss
150 8.1%)
Waste heatboiler loss
71 (3.8%)
Heat from cake combustion 1620 (87.3%)
1855 (100%)
Heat output by water in the cake
Heat output by burned gas arising from inflammable constituents of fuel and cake
848 (45.7%)
837 (45.1%)
To exhaust gas treatment facility 923 (49.8%)
Heat output by ash
20 (1.1%)
Heat from fuel combustion 175 (9.4%)
Recovered heat for drying 711 (38.3%)
(Material balance)
Dewatered cake 2083 (47.6%)
1294 (29.6%)
4373 (100%)
Dry gas 2449
(56%)
Dry air 701
(16%)
Water content 1098
(25.1%)
3063 (70%)
Fuel 16 (0.4%)
Evaporated water 789 (18%)
Air for combustion
Air for drying
Ash 125 (2.9%)
To exhaust gas treatment facility 4373 (100%)
Heat midium for drier

Fig. 23 Materials and heat balance (Cake moisture content of 80% and organic content of 70%; incinerator capacity of 50 ton/day)

Table 15 Comparisons of cake volume and heating value based on R-1

Model	R-1	R-2	R-3	R-4	R-5
$\frac{\text{Cake moisture content}}{\text{Cake volume of each model}}$ (%)	80.0	80.0	76.5	67.8	74.0
$\frac{\text{Cake volume of R-1}}{\text{Heating value of the cake}} \times 100$ (%)	100.0	100.0	85.2	62.1	76.9
Heating value of the cake (Kcal/kg)	246	246	394	762	500
$\frac{\text{Heating value of the cake of each model}}{\text{Heating value of the cake of R-1}} \times 100$	100	100	160	310	203

5.2 Cake Moisture Content and Auxiliary Fuel Requirement

The auxiliary fuel requirement of each model is shown in Fig. 24 under the condition that the furnace capacity is 50 (ton/day) and the cake organic content is 70%. As seen from this diagram, as the cake moisture content of each model decreases, the auxiliary fuel requirement decreases.

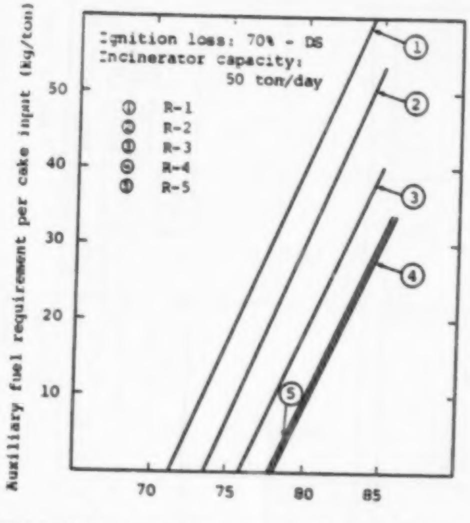


Fig. 24 Relationship between the cake moisture content and auxiliary fuel requirement

As shown in Table 16, the reduction in auxiliary fuel requirement in each model when the moisture content is reduced by 1% is almost the same providing that the organic content of the cake in each model is the same.

Table 16 Reduction in auxiliary fuel requirement by 1% reduction in the cake moisture content

Model / Organic content	R-1	R-2	R-3	R-4	R-5
60%	4.2	4.1	4.0	4.0	4.0
70%	4.7	4.7	4.6	4.5	4.5
80%	5.3	5.2	5.1	5.1	4.9

There is a tendency that when the organic content is high, the reduction in the auxiliary fuel requirement is large. The reduction in auxiliary fuel requirement when the organic content was 60% was about 4.0 (kg/ton), whereas the reduction was 5.1 (kg/ton) when the organic content was 80%. R-4 exhibits the least auxiliary fuel requirement when the water content is the same.

5.3 Cake Organic Content and Auxiliary Fuel Requirement

The auxiliary fuel requirement of each model, when the moisture content of the cake is 80% and the furnace capacity is 50 (ton/day), is shown in Fig. 25. As the organic content increases, the auxiliary fuel requirement of each model decreases.

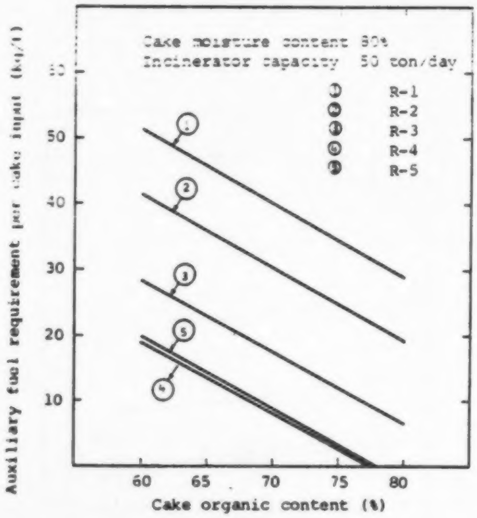


Fig. 25 Relationship between the organic content and auxiliary fuel requirement

As shown in Table 17, the reduction in auxiliary fuel requirement of each model by the increase in the organic content is the same providing that the moisture contents are the same. If the cake moisture content is at a low level, the reduction in the auxiliary fuel requirement becomes greater. The effect of the organic content on the auxiliary fuel requirement is smaller than that of the cake moisture content. Namely, a reduction in the auxiliary fuel requirement when the moisture content is reduced by 1% may correspond to the increase of the organic content by 4% to 5%.

Table 17 Reduction in auxiliary fuel requirement by 1.0% increase in the organic content

(Kg/t)

Model Cake moisture content	R-1	R-2	R-3	R-4	R-5
85%	0.84	0.83	0.83	0.82	0.83
80%	1.1	1.1	1.1	1.1	1.1
75%	1.4	1.4	-	-	-

5.4 Capacity of Furnace and Auxiliary Fuel Requirement

The auxiliary fuel requirement of each model, when the cake moisture content is 80% and the organic content is 70%, is shown in Fig. 26. As the capacity of the furnace increases, the auxiliary fuel requirement of each model decreases.

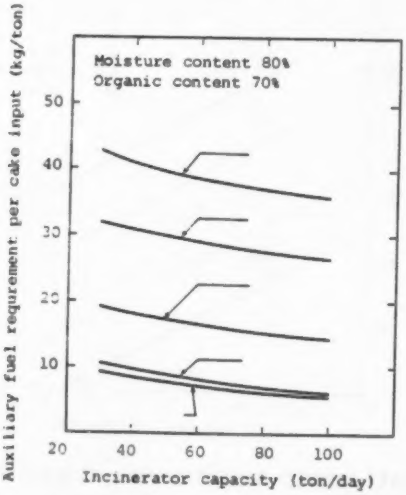


Fig. 26 Relationship between incinerator capacity and auxiliary fuel requirement

This is because the energy loss from the incinerator's main body in the heat balance computation is assumed to decrease in accordance with the total heat input as shown in Table 18.

Table 18 Heat loss ratio used for the calculation of the heat balance

Incinerator capacity (ton/day)	30	50	100
Heat loss ratio (%)	9.5	7.9	6.2

The reduction in the auxiliary fuel requirement of each model based on an incinerator with a capacity of 30 (ton/day) is shown in Table 19.

Table 19 Reduction in auxiliary fuel requirement by increasing the capacity of incinerator, based on the incinerator with a capacity of 30 (ton/day) (Cake with the moisture content of 80% and the organic content of 70%)

Model / Incinerator capacity	R-1	R-2	R-3	R-4	R-5
50 (t/day)	2.1	1.9	1.7	1.2	1.5
100 (t/day)	6.1	5.4	4.8	3.5	4.2

5.5 Condition for Autogenous Combustion

The result of the computation to seek for the moisture content that gives the limit for autogenous combustion is shown in Table 20.

Table 20 Critical moisture content for autogenous combustion (Incinerator capacity 50 ton/day)

(%)

Model / Organic content (%)	R-1	R-2	R-3	R-4	R-5
60	67.9	70.0	72.9	75.2	74.9
70	71.6	73.6	76.2	78.3	78.0
80	74.6	76.3	78.7	80.6	80.3

The heating values based on wet cake corresponding to the cake moisture contents shown in Table 20, are shown in Table 21.

Table 21 Critical heating value for autogenous combustion corresponding to moisture content shown in Table 20 (Incinerator capacity 50 ton/day)

(Kcal/kg-wet cake)

Model Organic content (%)	R-1	R-2	R-3	R-4	R-5
60	580	510	400	310	330
70	600	520	410	320	330
80	620	530	420	330	340

The minimum heating value that permits autogenous combustion is about 310 (Kcal/kg-wet cake), applicable to the case of R-4.

The boundary for autogeneous combustion each model is shown in Fig. 27.

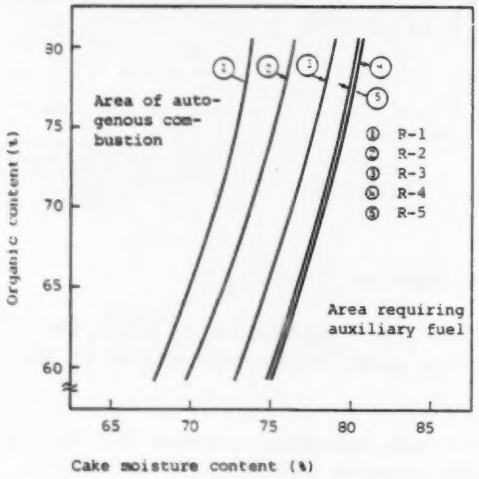


Fig. 27 Boundary of autogenous combustion for each model

To clarify the adaptability of each model with respect to the fluidized bed furnace, surveys are currently made to investigate the actual operating status of several fluidized bed reactors in service.

Tenth United States/Japan Conference
on Sewage Treatment Technology

THE PERFORMANCE OF THE CARVER-GREENFIELD PROCESS

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

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1. INTRODUCTION

The ward areas are divided into ten drainage areas owing to the topographical features such as rivers and ground elevations.

Under the present master plan for sewerage in the ward areas, 10.4 million people will be provided with a sewerage system over a projected area of 53,827 ha when completed. General facts and figures for sewerage planning in the ward area are shown in Table - 1 and Fig. - 1 shows the location of treatment plants and drainage areas.

The ratio of sewered population in the ward areas rose to 80% by the end of fiscal 1983.

2. PRESENT CONDITION OF SLUDGE TREATMENT AND DISPOSAL

The average volume of daily sewage accepted by the nine plants in the 23 wards was about 4.4 million cubic meters in fiscal 1983.

And the average volume of daily raw sludge was about 95,600 cubic meters.

Table - 2 shows the statistics of sewage and sludge treatment in Tokyo's Ward Area in fiscal 1983.

In future, Central sludge handling stations for the processing of sludge collected by pressure conduit, are planned to be built around reclaimed land in Tokyo Bay in order to meet the demand for sludge treatment. The Nanbu sludge treatment plant is already in operation.

Mechanically dewatered sludge cake is now mainly disposed to Sanitary landfill in Tokyo Bay. But the sites available for landfill will be limitted, so about 70% of cake is incinerated to reduce its volume. In fiscal 1983, the average volume of $908m^3$ of dewatered sludge, 133t incinerated ash, and 7t of Alumina cement were kneaded for stable landfill. On the other hand, development of new technology for reuse of sludge is in under way.

3. TRIAL OF EFFECTIVE REUSE OF SLUDGE

(1) Compost

Municipal sludge contains essential plant nutrients and useful trace elements and therefore has potential use as a fertilizer or soil conditioner. Liquid sludge will be available if the cropland were far from residential area, but in Tokyo, those are close together so the dewatered cake is fermented by aerobic bacteria. The composted sludge is rather stable, free of pathogenic problem and odor and is easy to handle.

The composting plant in operation from 1978 at Minamitama treatment plant produces two or three tons of composted sludge from the seven tons of dewatered sludge cake daily. The product is sold to farmers for a fertilizer in vegetable fields or as a soil conditioner. Fig.-2 is the flow diagram for the process.

(2) Artificial lightweight aggregates

One way to utilize inorganic matter in sludge is to make artificial light-weight aggregates from sludge ash.

The basic technology for this process has already been successfully developed and manufacturing plant was constructed at the Odai treatment plant in 1983.

This plant has a daily production of 12 metrictons.

The quality of artificical lightweight aggregate is sufficient to replace natural lightweight aggregates and some of the uses for flower pots and building materials, etc. were investigated.

Fig.-3 shows the flow diagram of this process.

(3) Sludge fuel

A large pilot plant was installed at the Sunamachi treatment center for the purpose of applying the multiple-effects evaporation process (CG process) for the extraction of water from the dewatered sludge cake in 1983.

The water extraction capacity is about 200kg per hour.

Dried sludge has many practical uses and one of them is as a fuel to produce steam for power generation.

(4) Smelting slag

Techniques for utilization of the energy in dried sludge have also been studied. If the dried sludge is heated over its melting point (about 1400 to 1500°C), and then cooled, its volume can be reduced considerably. And the waste heat can be effectively used for the drying of sludge cake.

In 1984, a pilot plant with a daily processing of 50kg of dried sludge, was built at the Sunamachi treatment center for performing experiments for drying, melting and crystallization process technology, and for the durability of the facilities.

In order to develope effective technology in the reuse of sewage sludge, many efforts have been done in Tokyo. Among them, some information about sludge fuel system are as follows.

4. OUTLINE OF SLUDGE FUEL SYSTEM

(1) The intention of the project

Sewage treatment is the separation process of solid and water and also such is the same in sludge treatment processes.

So improvement in techniques in removing water from solids is the key point. In this point of view, at first, the nature of raw sludge should be investigated carefully, and the unit processes that fit for the raw sludge, must be carefully adopted.

Owing to the changes of life style of citizens in Tokyo, the ratio of organic materials in raw sludge has gradually increased.

At present, about 70% of solid in sludge, is organic matter.

So the activated sludge derived from final sedimentation tanks, is very difficult in gravity thickening.

Improving in sludge thickening process is one point.
Research for improving gravity thicknor, dissolved air floatation, and centrifuge are now undergoing.

The next is mechanical dewatering process. Performance of vacuum filter, press filter, centrifuge and belt press filter are investigated.

Now, belt press filter, with the aid of polymer dosage, is becoming popular.

Third point is the process of incineration Multi-staged incinerator, and fluidized sand bed incinerator, equipped with anti-air pollution devices, are in use.

The aim of improving incineration process is autogenous combustion without addition of supplement fuel.
For the treatment of special sewage sludge, reasonable combination of unit processes should be applied.

In this way, the most economical sludge treatment process will be established in reducing the volume of sludge disposal.

Even if the volume of disposal would be small, the sites for landfill will be needed forever.

On the other hand, organic substances in the sludge solids will have nutrients for plant and also calorific values to some extent.

So development of technology in reusing sewage sludge is very important.

Research for effective reuse of sewage sludge will be divided into three types owing to the nature of sludge.

First is to use nutrient as fertilizer. Because of the urbanization of Tokyo area, the constant need for compost will be limitted.

Second is to use the solids of sludge as a construction materials such as artificial light-weight aggregates and artificial slags.

Third is to use calorific substances in the solids as fuels.

Anaerobic digestion of thickened sludge has been emploied in many sewage treatment plant in Tokyo. And methane gas produced in digestor is now used for heating boiler. But about the half of organic materials remain in digested sludge for next treatment.

Recovering the all organic substances in sludge as fuels is sludge fuel system.
In sludge fuel system, the most effective and economical removing of water from solids is expected.

Multi-effect evaporation processes have been successfully applied in many food and chemical industries with less energy consumptions.

However, this process is not applicable to slurries such as sludge, because of its scaling and plugging in heat exchangers.

The carver-Greenfield process, using carrier oil for preventing scaling, is the one that is expected efficient evaporation of water from a slurry like sewage sludge.

In Japan three-staged multiple-effect evaporation system was introduced at city of Fukuchiyama in Kyoto prefecture in 1976.

Therefore, Tokyo installed experimental plant in 1982, in order to investigate the performance of multiple-effect evaporation process with the dewatered sludge instead of the thickened sludge.

And also studies for using sludge fuels produced as a fuel and a raw materials in cement manufacturing industry and for fuels in power generation plant are now implementing.

(2) Outline of the pilot plant

The key to energy recovery from high-moisture organics such as sewage sludge, without use of supplemental fuels, is the energy-efficient removal of water prior to combustion.

A number of processes for industry have been developed to separate water from solids with less energy input than conventional heat drying systems.

In Japan Fukuchiyama city is now operating the three-staged multiple-effect evaporation facilities for drying the thickened activated sludge before combustion.

Fig.-4 shows the flow diagram of sludge fuel system.

Table-3 shows an outline of the pilot plant implemented in the field demonstrations and engineering studies.

Flow sheet, layout, sections of the plant are expressed in Fig.-5.6.7.

(3) Results of studies

a) Supplied dewatered sludge

Thickened the composite of raw and activated sludge is dewatered by belt press dehydrator.
The natures of sludge cake are shown in Table-4.

b) Fluidizing oil

Dewatered sludge is first mixed with oil.
The carrying oil assures the maintenance of fluidity in all phases of evaporation cycle and prevention of scaling, plugging of the heat exchangers.
The characteristics of carrying oil should be easy in mixing with sludge, and maintaining fluidity in all phases.
Also the mixture must be pumped out easily.

For this purpose, the viscosity of oil and fiber component in sludge are very important. Heavy oil is selected and confirmed the effect.
Fig.-8 shows the viscosity of various oil correspond to temperature.
In summer, being fiber-component in sludge 2 or 3 percent (dry base), 6 times of oil against the weight of solids in sludge is needed.
Meanwhile in winter, 8 times of oil is needed owing to the 10% of fiber in sludge.

c) Multiple-effect evaporator

Sludge-oil slurry is then pumped to four-staged multiple-effect evaporator where vapor from the previous effect vaporizes the water in the sludge.
Fig.-9 and Fig.-10 show the performance of four-staged multiple-effect evaporator in temperature and pressure.
In order to evaluate the performance of the evaporator, overall heat transfer coefficient, efficiency of evaporation, and multiple of evaporation are expressed.

For the ability of evaporator, the rate of heat transfer q [Kcal/hr] is expressed as follows.

$$q = A, u, \Delta t$$

where A: Areas of heat transfer (m^2)
u: Overall heat transfer coefficient ($Kcal/m^2.hr.°C$)
Δt: Difference in temperature between vapor and liquid (°C)

The overall heat transfer coefficient means the quantity of heat transfered per unit area. Therefore it plays a important role in scaling-up the facilities.

Table-5 shows the coefficient observed in this pilot plant. On the other hand Table-6 shows that of Fukuchiyama. Depending on the fluidity of slurry the coefficient of the fourth-stage of Tokyo is equivalent to that of Fukuchiyama. The economical aspect of the evaporator is referred to efficiency of evaporation and multiple of evaporation. The efficiency of evaporation 7(%), and multiple of evaporation $a\left(\frac{\text{water-kg/}}{\text{/steam-kg}}\right)$ are expressed as follows.

$$\eta = \frac{W}{N \times S} \times 100$$

$$a = \frac{W}{S} = \frac{N.\eta}{100}$$

where W: Quantity of evaporated water (Kg/hr)
N: Number of evaporation effect
S: Quantity of steam supplied (Kg/hr)

According to this pilot plant study, 2.6kg of water is expected to be evaporated for each kg of steam supplied.
The efficiency of evaporation for this plant is 65%.
Meanwhile, that of Fukuchiyama is reported 75%.
As shown in Fig.-10, energy efficiency for vaporizing unit weight of water will be declining owing to the quantity of oil in slurry supplied.

d) Distillate

The vaporized water is condensed and discharged. The quality of distillate is shown in Table-7.
It contains steam distillable and water soluble substances which can be readily biodegraded by an activated sludge process.
The increase in quality and quantity of inflow sewage at the treatment plant by mixing the distillate will be reached few percent.

e) Solids and oil separation

The remaining solid-oil mixture from the fourth-stage effect contains about 80 percent of oil.
The carrying oil is separated from solids using screw-press, and will be recycled for reuse in the evaporative cycle.
Table-8 shows the results of operation of screw-press.
Oil and grease content of the dried sludge products is about 15 or 20 percent (dry base). While, the average content of n-hexane extractables in original sludge cake is about 11 percent. So residual fluidizing oil in dried sludge products will be under 10 percent.

City of Los Angels is now introducing thermal process in sludge treatment of Hyperion sewerage plant.
Digested centrifuged sludge cake is mixed with light solvent oil such as AMSCO140 and then pumped to four-staged multiple-effect evaporator, where water is vaporized. The remaining solid-oil mixture is then centrifuged and hydroextracted to separate carring oil from solids.

f) Combustion test

An extensive fluidized bed incineration test for sludge fuel was conducted. The objectives of the test program were to develop detailed criteria for design and air emission factors for subsequent air permit applications.
Major conclusion and recommendations from the test program are summarized as follows.
Sludge fuel can be combusted effectively and organic content of incinerated ash is below 0.5 percent.
The effectiveness of incineration will improve with the increase of the fluidizing air, but significant differences in combustion characteristics were not observed.
With the increase in feeding load of sludge fuel, the effectiveness of incineration will be decline, and the temperature will go up.
In this test, feeding load of 800 x 10^3Kcal/m^2.h is optimum.
NOX emission is related to exhaust gas O^2 content, and can be controlled by regulating the O^2 content.
About 3% of fuel bound nitrogen will be converted to NOX, under the air supply of 100 percent.
About, 80 percent of fuel bound sulfur will be converted to SOX.
As far as HCN and colour, no problem is observed in this test.

5. FULL-SCALE MODEL PLANT

Full-scale pilot plant with the capacity of 50t-DS/day will be installed at Nanbu sludge treatment site.

In this plant, further experiments for the performance of multiple-effect evaporation system, screw press, and power generator will be studied.

Table 1. General Facts and Figures on Planning for Tokyo's Sewerage System (for the ward area)

(City plan at end of March 1984)

Name of drainage area	Projected sewered population	Projected sewered area (ha)	Trunk sewers (m)	No. of pumping stations	Treatment plant	
					No.	Capacity (m^3/day)
Total	10,358,000	53,827	1,099,910	68	15	9,970,000
Shibaura	883,000	6,420	149,350	13	1	1,590,000
Mikawashima	974,000	3,936	106,440	10	3	950,000
Sunamachi	957,000	4,309	95,000	16	1	1,220,000
Odai	371,000	1,687	23,250	4	2	420,000
Ochiai	678,000	3,506	52,090	-	2	590,000
Morigasaki	2,324,000	12,882	265,860	9	1	1,810,000
Kosuge	323,000	1,633	28,740	3	1	450,000
Kasai	993,000	4,540	83,250	8	1	940,000
Shingashi	1,980,000	10,474	186,390	1	2	1,390,000
Nakagawa	875,000	4,440	109,490	4	1	610,000

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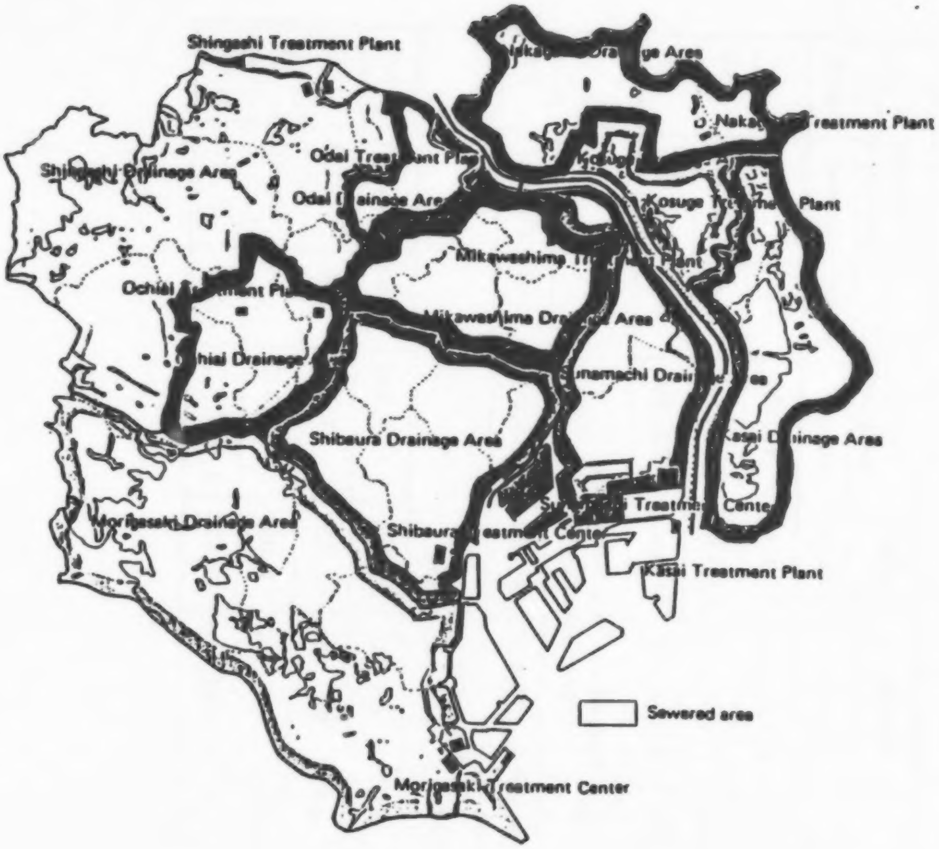

Shingashi Treatment Plant
Nakagawa Drainage Area
Nakagawa Treatment Plant
Odai Treatment Plant
Shingashi Drainage Area
Kosuge
Odai Drainage Area
Kosuge Treatment Plant
Mikawashima Treatment Plant
Ochiai Treatment Plant
Mikawashima Drainage Area
Ochiai Drainage
Sunamachi Drainage Area
Kasai Drainage Area
Shibaura Drainage Area
Sunamachi Treatment Center
Morigasaki Drainage Area
Shibaura Treatment Center
Kasai Treatment Plant
Sewered area
Morigasaki Treatment Center

Figure 1. Total Planning for the Ward Areas' (At end of March 1984) Sewerage System

Table 2. Statistics of Sewage and Sludge Treatment in Tokyo's Ward Area (At end of March 1984)

Treatment Plant	Volume of Sewage Treatment(m^3)		Volume of Sludge Treatment(m^3)		Volume of Dewatered(t)		Volume of Incineration(t)	
	Annual	Daily	Annual	Daily	Annual	Daily	Annual	Daily
Shibaura	274,320,080	749,520	2,624,780	7,170	92,801	250		
Sunamachi	182,011,370	497,300	7,660,250	20,930	311,622	850	311,622	851
Morigasaki	410,017,830	1,120,270	9,994,640	27,310	312,840	860	112,117	306
Mikawashima	222,425,480	607,720	Pumped to the Sunamachi treatment center					
Odai	107,181,550	292,850	8,829,470	24,120	235,096	640	204,637	559
Ochiai	165,890,250	453,250	Pumped to the Odai treatment plant					
Shingashi	153,823,820	420,280	4,306,390	11,770	77,633	210	71,568	196
Kosuge	56,123,700	153,340	847,010	2,320	17,151	50	17,151	47
Kasai	33,412,280	91,290	733,270	2,000	15,330	40	14,202	39
Total	1,605,209,350	4,385,820	34,995,810	95,620	1,062,473	2,900	731,297	1,998

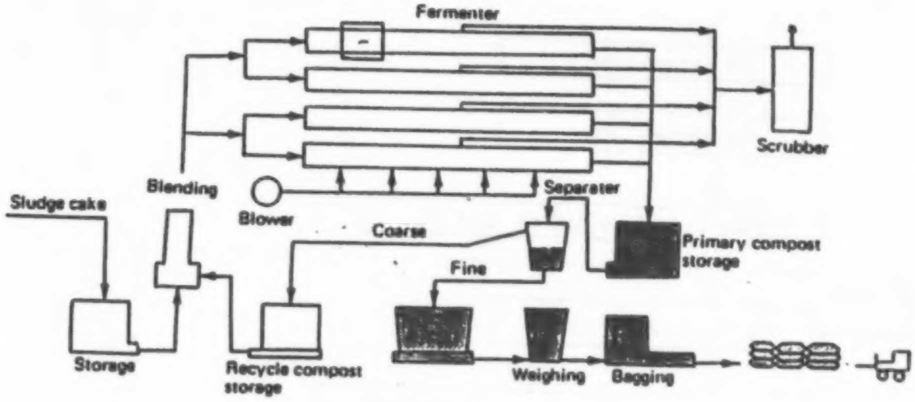

Fermenter
Scrubber
Blending
Sludge cake
Blower
Separator
Coarse
Fine
Primary compost storage
Storage
Recycle compost storage
Weighing
Bagging

Figure 2. Flow Diagram of Composting Process at Minamitama

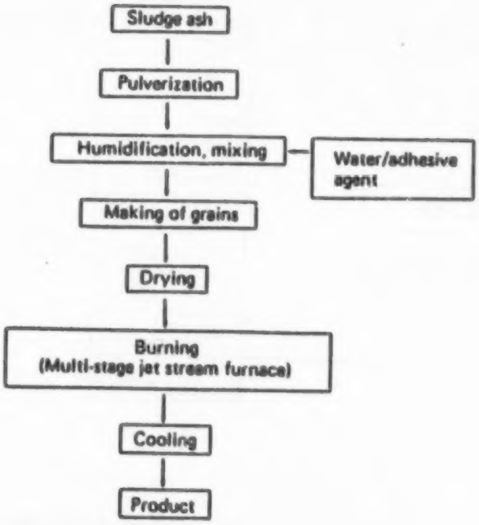

Sludge ash
Pulverization
Humidification, mixing
Water/adhesive agent
Making of grains
Drying
Burning
(Multi-stage jet stream furnace)
Cooling
Product

Figure 3. Flow Diagram of the Artificial Lightweight Aggregate Manufacturing Process

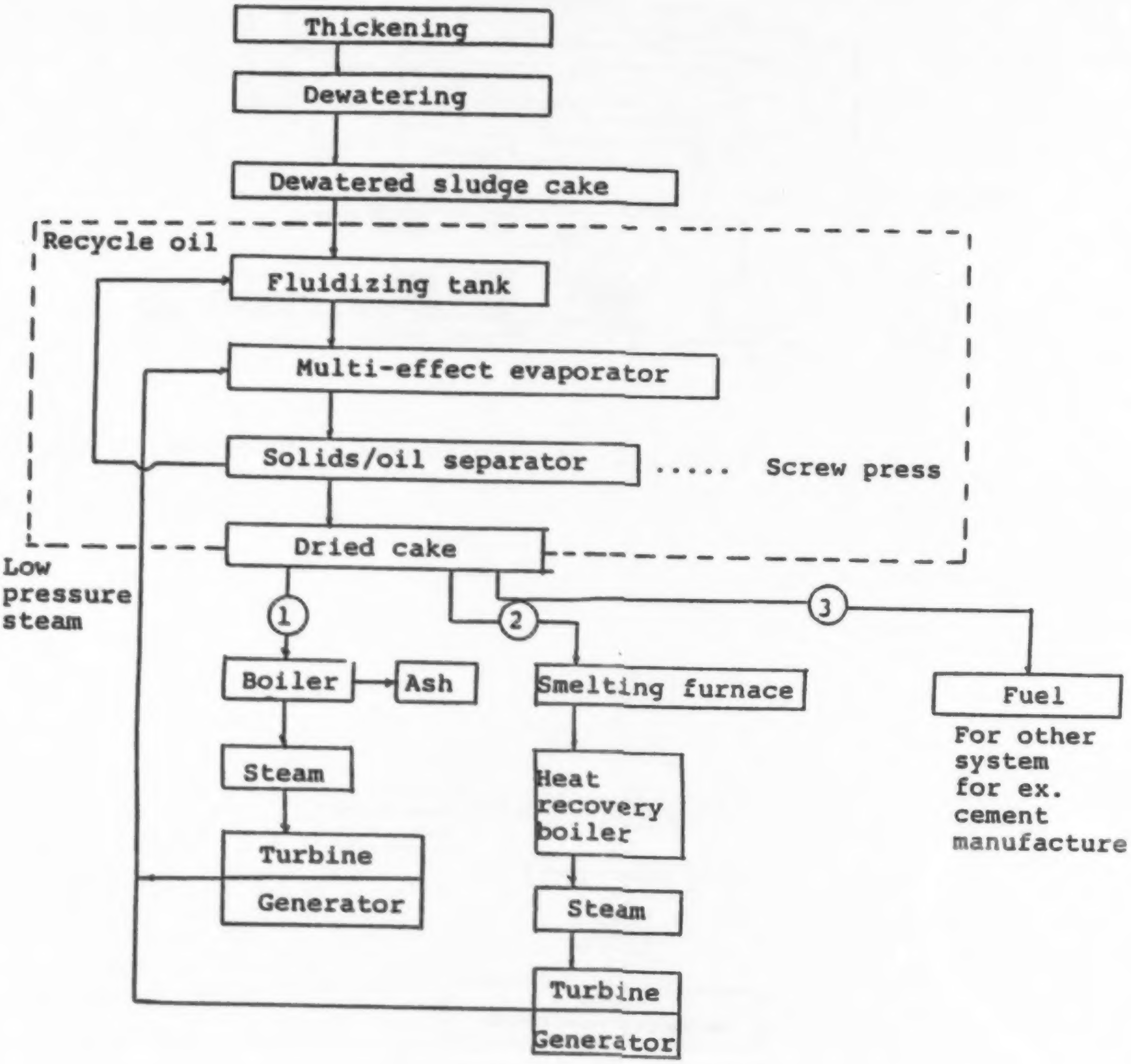

Thickening
Dewatering
Dewatered sludge cake
Recycle oil
Fluidizing tank
Multi-effect evaporator
Solids/oil separator
..... Screw press
Dried cake
Low pressure steam
1
2
3
Boiler
Ash
Smelting furnace
Fuel
For other system for ex. cement manufacture
Steam
Heat recovery boiler
Turbine
Generator
Steam
Turbine
Generator

Figure 4. SF-system Diagram

Table 3. An Outline of The Pilot Plant

1. Process Design Specifications

Dewatered sludge cake input (moisture content 80%)	250 kg/h
Dry solids input	50 kg/h
Condensed vapors output	195 kg/h

2. Major Equipment Specifications

(1) Multiple effect evaporator

1) Heat exchanger	x 4	
o Heating surface area	6.25 m^2/set x 4 sets = 25.0 m^2	
o Dimensions	Diameter	320 mm
	Height	4,200 mm
2) Evaporator	x 4	
o Dimensions	Diameter	270 mm
	Height	4,000 mm
3) Condenser	x 1	
o Heating surface area	6.25 m^2	
o Dimensions	Diameter	320 mm
	Height	4,000 mm

(2) Solids/oil separator	x 1	
Type	Screw press	
Capacity	50 kg-DS/h	
Dimensions	Diameter	300 mm
	Length	3,000 mm

(3) Boiler	x 1
Type	Package boiler
Fuel	Heavy oil
Steam capacity	Max. 200 kg/h
Steam pressure	Max. 10 kgf/cm^2 gage

(4) Mixed slurry feeder

1) Fluidizing tank	x 1	
o Dimensions	Diameter	970 mm
	Height	2,000 mm
o Capacity		1,000 kg
o Retention time		20 minutes
o Power required		3.7 kW
2) Mixed slurry feed tank	x 2	
o Dimensions	Diameter	1,940 mm
	Height	3,000 mm
o Capacity		5,000 kg

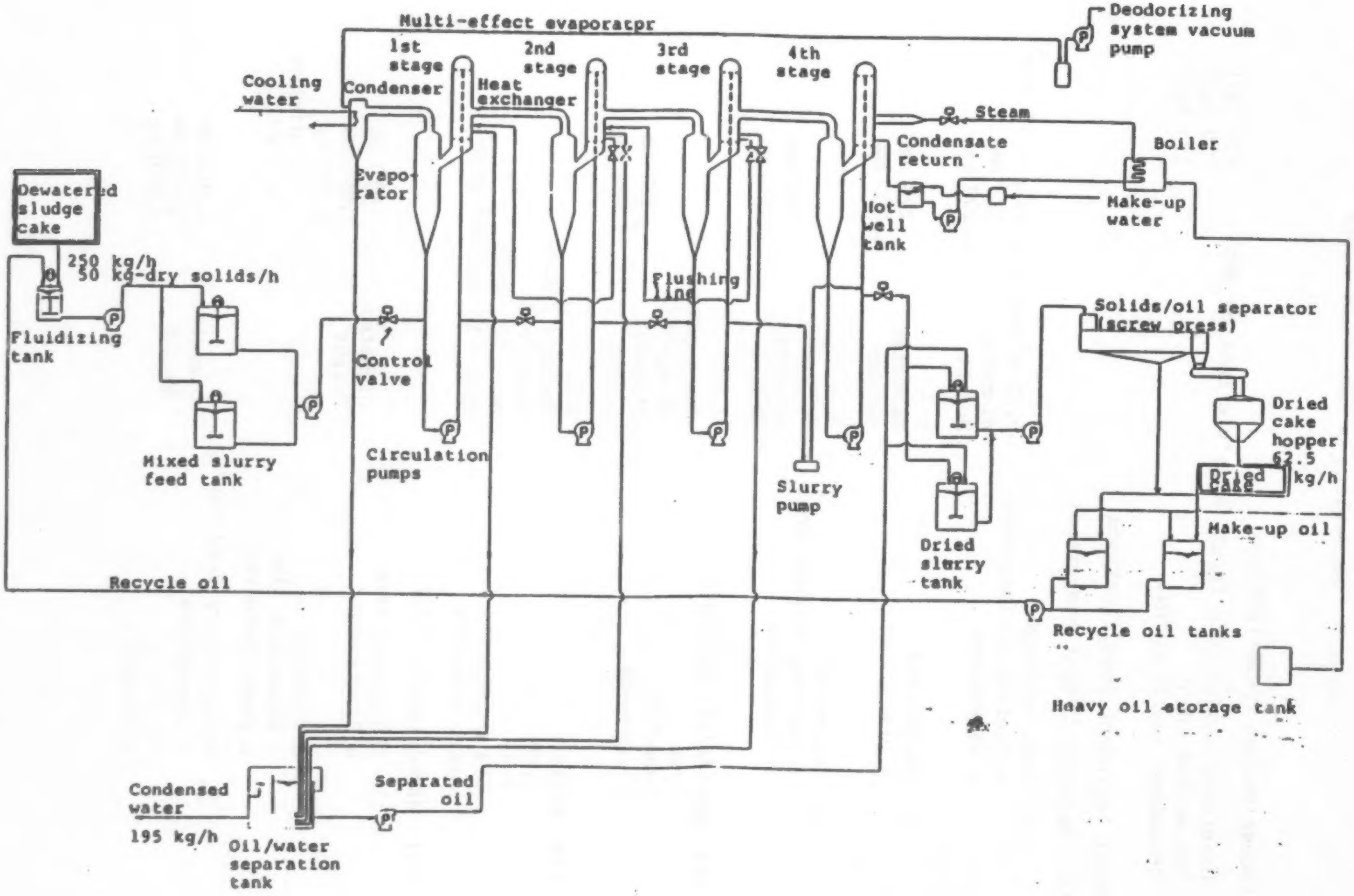
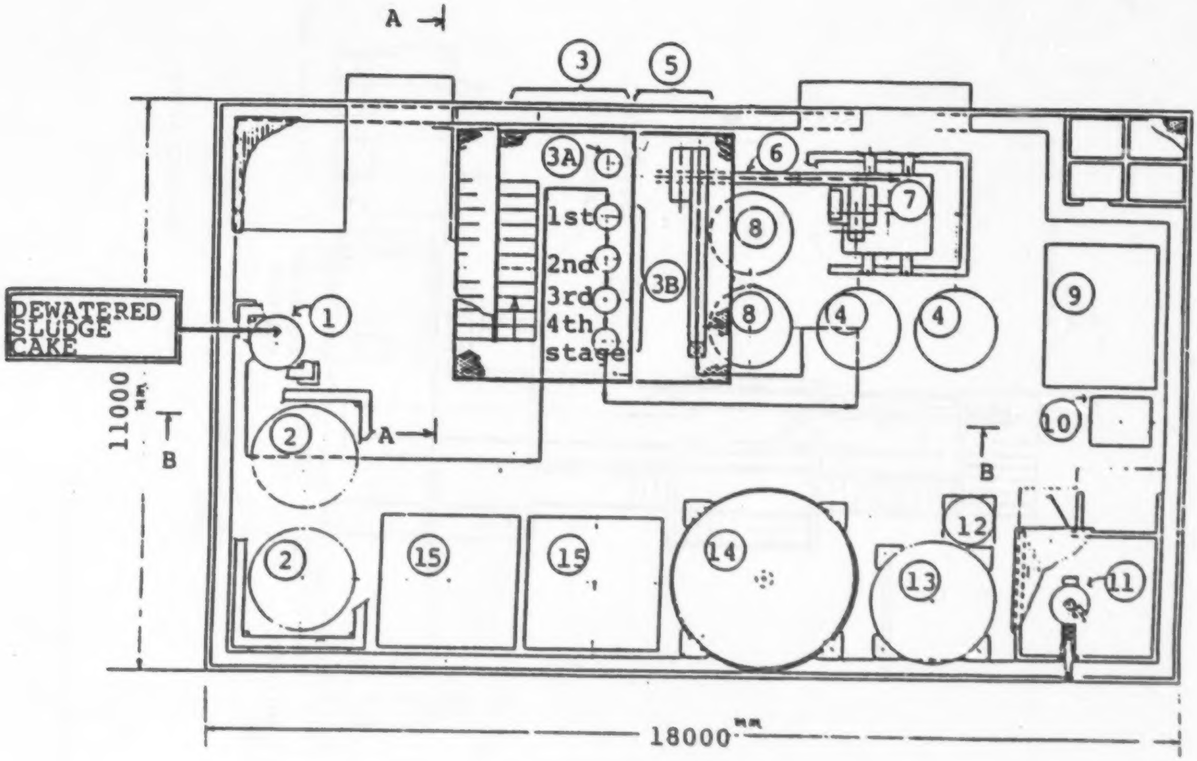

Multi-effect evaporatpr
Deodorizing system vacuum pump
1st stage
2nd stage
3rd stage
4th stage
Cooling water
Condenser
Heat exchanger
Steam
Condensate return
Boiler
Dewatered sludge cake
Evapo-rator
Hot well tank
Make-up water
250 kg/h
50 kg-dry solids/h
Flushing line
Solids/oil separator (screw press)
Fluidizing tank
Control valve
Dried cake hopper
62.5 kg/h
Dried cake
Mixed slurry feed tank
Circulation pumps
Slurry pump
Dried slurry tank
Make-up oil
Recycle oil
Recycle oil tanks
Heavy oil storage tank
Condensed water
195 kg/h
Separated oil
Oil/water separation tank

Figure 5. Pilot Plant Flow Sheet


A
3
5
3A
6
7
1st
2nd
3rd
4th
stage
8
3B
8
4
4
9
DEWATERED SLUDGE CAKE
1
11000 mm
B
2
A
10
B
12
2
15
15
14
13
11
18000 mm

① Fluidizing tank
② Mixed slurry feed tanks
③ Multi-effect evaporators
  3A Condenser
  3B Heat exchangers
④ Dried slurry tanks
⑤ Solids/oil separator
⑥ Dried cake conveyor
⑦ Dried cake hopper

⑧ Recycle oil tanks
⑨ Deodorizing system
⑩ Heavy oil storage tank
⑪ Boiler
⑫ Boiler make-up water tank
⑬ Cooling water storage tank
⑭ Gravity thickener
⑮ Sludge storage tank

Figure 6. Pilot Plant Layout

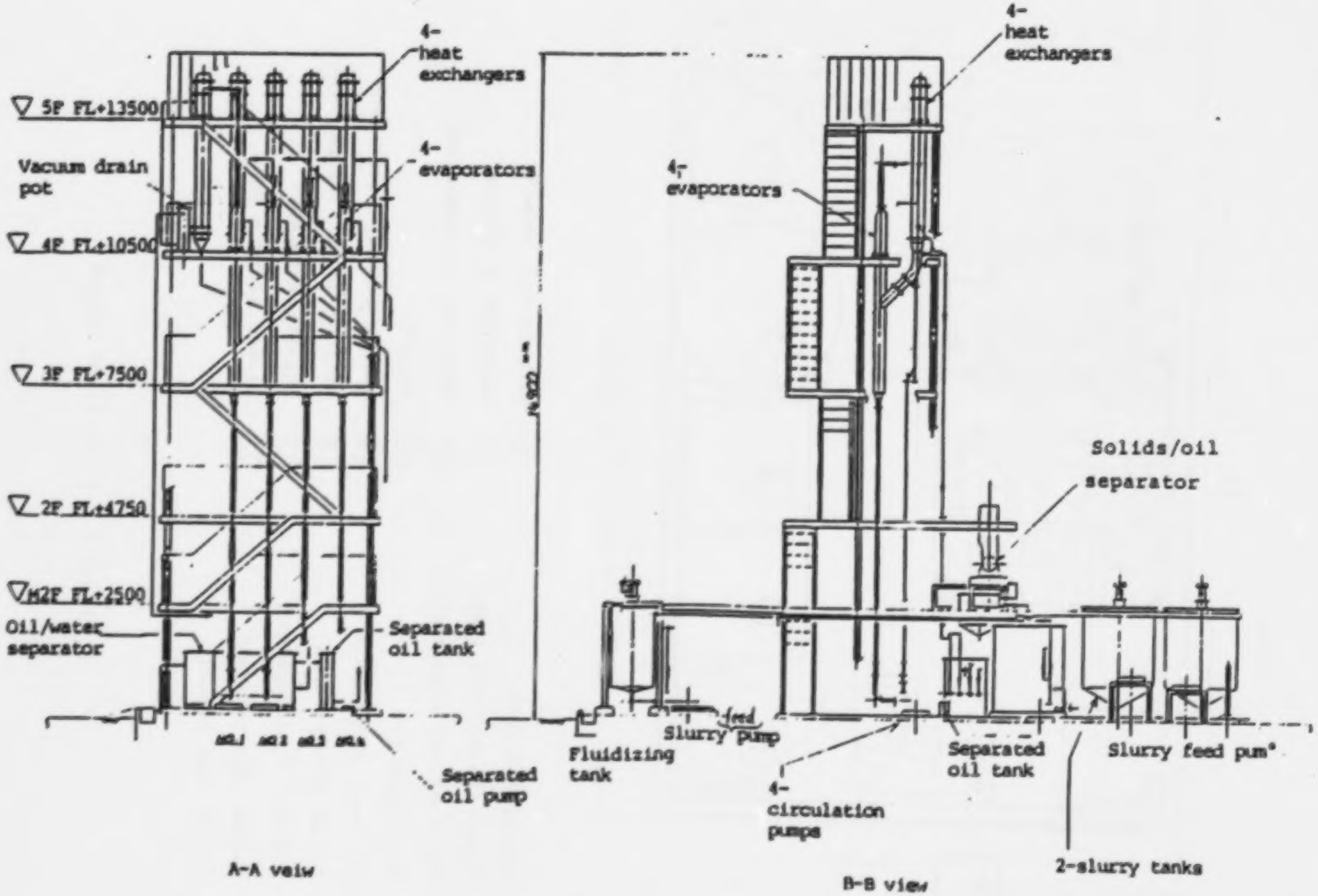

4-
heat
exchangers
4-
evaporators
Vacuum drain
pot
5F FL+13500
4F FL+10500
3F FL+7500
2F FL+4750
M2F FL+2500
Oil/water
separator
Separated
oil tank
Separated
oil pump
A-A veiw
4-
heat
exchangers
4-
evaporators
Solids/oil
separator
Fluidizing
tank
Slurry feed pump
4-
circulation
pumps
Separated
oil tank
Slurry feed pum'
2-slurry tanks
B-B view

Figure 7. Pilot Plant Section

Table 4. Nature of Sludge Cake

		PH	Mois- ture content	Furnace reduct- ion	N-hexane extracts	NH_3-N	Volatile organic acid	Metal soap	Coarse sus- pended solid
		(-)	(%)	(%-DS)	(mg/g-DS)	(mg/g-DS)	(mg/g-DS)	(%-DS)	(%-DS)
1983	3/10	7.4	74.9	77.6	(36.4)	17.3	52.9	-	
	3/17	7.9	72.8	74.5	(30.5)	12.5	50.2	-	
	3/18	7.6	71.1	74.6	(25.8)	11.0	58.1	-	
	3/23	7.2	74.5	75.0	(31.9)	12.2	54.9	-	
	3/28	6.2	73.6	77.3	(31.4)	9.1	64.5	0.79	
	3/29	6.1	72.7	77.6	(40.1)	9.2	33.1	-	
	5/11	6.8	74.5	78.0	(30.4)	3.4	93.5	0.74	
	5/30	-	78.7	-	-	-	-	1.49	
	8/ 2	-	80.6	-	(20.4)	-	-	3.94	
	8/ 8	-	79.7	-	(27.4)	-	-	2.41	
	8/23	-	78.4	74.1	97.8	12.3	89.8	-	6.5
	9/ 1	-	79.0	-	91.8	-	-	2.97	
	9/ 6	5.5	79.3	73.2		6.8	66.9	-	6.6
	9/13	-	79.0	-	92.0	-	-	1.45	
	9/21	-	76.3	71.2	-	-	-	-	
	9/26	-	76.9		-	6.1	65.8	-	9.4
	9/27	-	75.6		107.0			-	
	10/ 6	-	76.8	64.3	119.0	-	-	2.01	16.0
	10/13	-	79.8	70.4	-	3.7	71.7	-	
	10/18	-	76.9	75.9	126.0	-	-	1.97	
	11/ 2	5.7	76.9	76.3		3.6	56.4	-	16.8
	11/11	5.6	80.7	78.0	91.9	3.8	63.1	1.45	14.2
	11/15	5.2	76.8	79.2	88.1	-	-	-	
	11/24		77.0	77.6	96.7	-	-	1.63	23.2
	11/28		79.3		-	5.4	86.2	-	
	12/14	5.5	79.1	77.8	14.4	5.9	51.8	1.42	18.7
1984	1/17	5.8	80.6	77.8	-	6.5	83.3	-	14.5
	2/21	-	77.3	74.1	10.5			3.12	16.5
	3/23	6.0	78.1	65.7		4.5	60.9	2.21	13.9

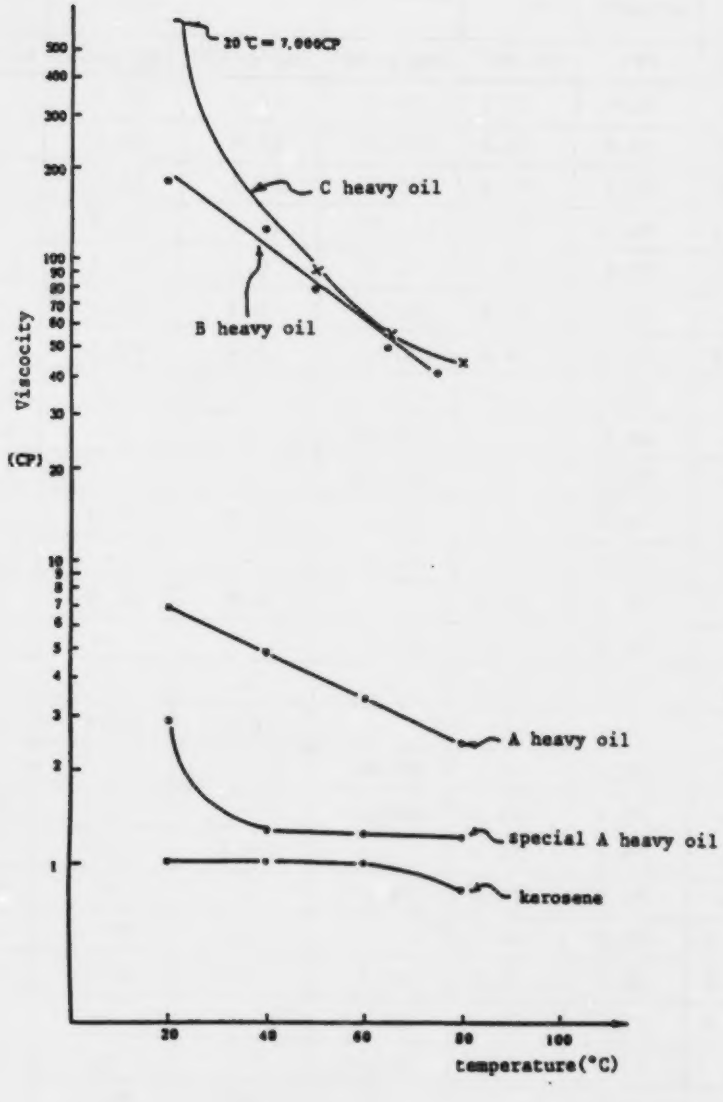


Figure 8. The Viscocity of Heavy Oils Correspond to Temperature

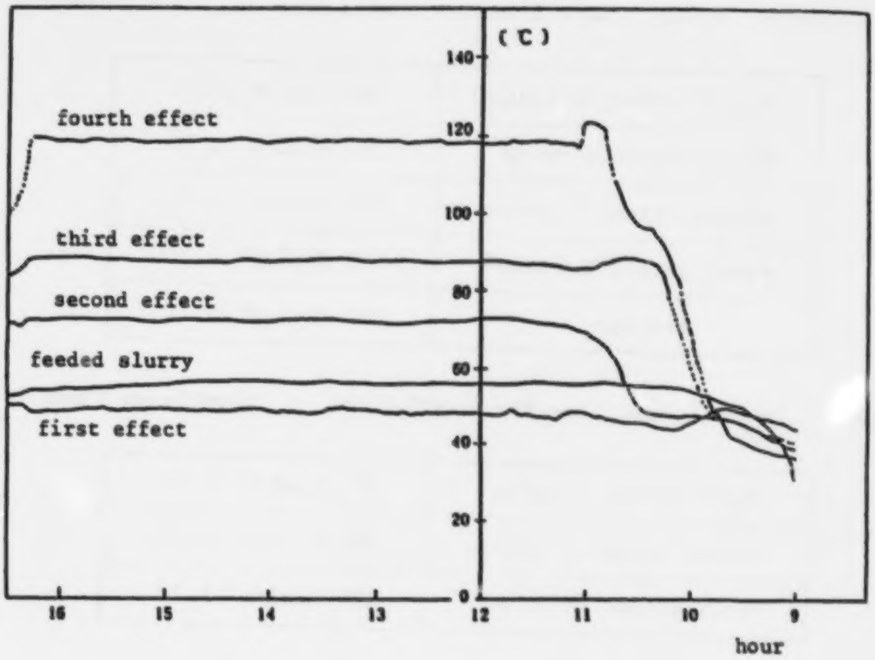


Figure 9. The Temperature of Evaporator

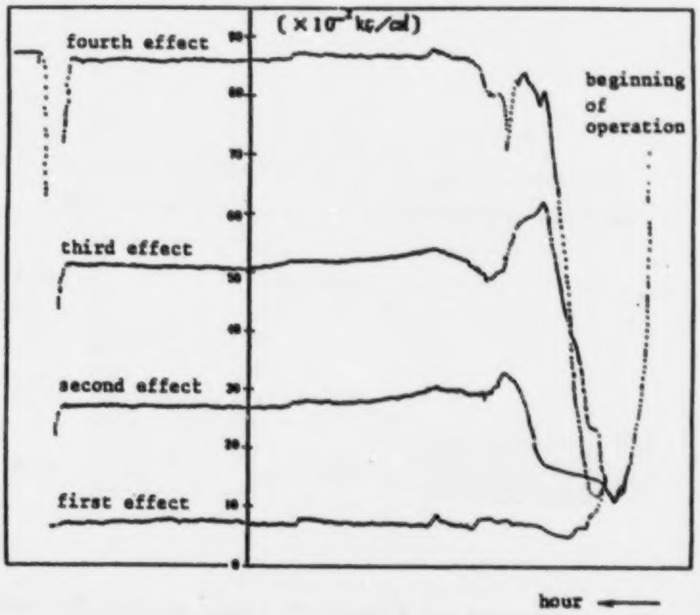


Figure 10. The pressure of Evaporator

Table 5. Overall Heat Transfer Coefficient of Pilot Plant

Fourth effect U value	290 Kcal/m^2.h.°C
Third effect U value	260 Kcal/m^2.h.°C
Second effect U value	240 Kcal/m^2.h.°C
First effect U value	170 Kcal/m^2.h.°C
Average	240 Kcal/m^2.h.°C

Table 6. Overall Heat Transfer Coefficient of Fukuchiyama Plant

Third effect U value	290 Kcal/m^2.h.°C
Second effect U value	280 Kcal/m^2.h.°C
First effect U value	280 Kcal/m^2.h.°C

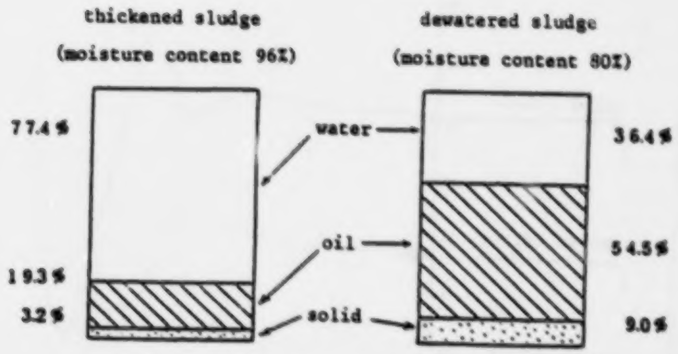


Figure 10. Component of Slurry (6 times of oil for solids)

Table 7. Quality of Distillate

PH	9.4
SS	11.5 mg/ℓ
TS	123 mg/ℓ
COD_{Mn}	1,740 mg/ℓ
BOD_5	11,890 mg/ℓ
NH_4 - N	2,940 mg/ℓ
Volatile organic acid	6,080 mg/ℓ

Table 8. Results of Operation of Screw-Press

Operation date	Operational condition						Quantity		Dried cake					Recovered oil						Ratio of recovered SS
	Number of rotation	Taper corn	Slurry feed				kg/h	kg-DS/h	Solids	Moisture content		Oil content		Moisture content	Solids	Viscocity				
			Slurry q'ty	Solid	Rates of oil for solids	Temperature										20°C	4C°C	60°C	80°C	
	rpm	mm	kg/h	kg/h	-	°C			%	%	%-DB	%	%-DB	%	%					%
1983 Nov.25	0.5	39~42	437	49	7.9	120	45.6	37.8	83.0	2.5	3.0	14.5	17.5	0.4	3.5	33.5	19.3	8.8	5.8	77.1
Nov.30	0.5	36	460	53	7.7	120	48.5	39.6	81.7	3.0	3.7	15.3	18.7	0.5	4.8	28.4	14.2	8.1	5.8	74.8
Dec. 2	0.5	36	460	51	8.0	120	47.5	38.1	80.2	3.8	4.7	16.0	20.0	0.9	5.2	23.7	13.2	7.8	5.1	74.7
Dec. 7	0.8	40	381	40	8.5	120	39.5	29.9	75.6	4.1	5.4	20.3	26.9	0.7	5.8	26.7	13.9	8.8	-	74.8
Dec. 9	0.8	35	409	44	8.3	120	40.4	32.5	80.5	4.7	5.8	14.8	18.4	0.5	5.1	26.4	13.9	7.4	5.4	73.9
1984 Jan.18	0.8	40	349	41	7.5	115	37.1	30.4	82.0	3.4	4.2	14.6	17.8	0.5	6.3	38.9	14.9	7.8	4.9	74.2
Jan.24	0.8	36	388	44	7.9	117	43.9	35.1	80.0	2.5	3.1	17.5	21.9							79.8
Jan.27	0.8	36	355	41	7.7	115	34.8	27.9	80.1	3.5	4.4	16.4	20.5	0.4	3.4	39.9	16.2	8.1	5.4	68.0
Feb. 1	0.8	38	470	55	7.5	115	46.2	37.1	80.2	3.6	4.5	16.2	20.2	0.6	5.3	25.7	14.9	8.1	5.4	67.4
Feb. 6	0.8	38	491	52	8.4	114	48.6	39.7	81.6	3.2	3.9	15.2	18.6	0.5	2.2	33.5	15.9	8.1	5.8	76.3
Feb. 9	0.8	40	434	49	7.8	115	47.7	39.0	81.7	3.3	4.0	15.0	18.4	0.4	3.3	33.2	16.6	8.8	5.6	79.5
Feb.13	0.8	36	494	58	7.5	114	52.2	41.0	78.5	2.5	3.2	19.0	24.2	0.5	5.7	30.5	16.6	8.8	5.4	70.7
Feb.16	0.8	38	481	56	7.6	113	48.4	39.1	80.8	3.0	3.7	16.2	20.0	0.4	4.8	32.8	15.2	8.5	5.4	69.8
Feb.20	0.8	36	407	44	8.2	114	37.8	31.2	82.5	3.2	3.9	14.3	17.3	0.5	5.8	30.1	15.2	8.1	5.4	70.9
Feb.22	0.8	38	478	53	8.0	114	55.3	44.5	80.5	4.0	5.0	15.5	19.3	0.5	5.1	27.8	13.9	8.1	5.1	84.0
Feb.24	0.8	40	427	50	7.6	115	47.4	38.6	81.5	4.0	4.9	14.5	17.8	0.5	3.7	31.1	15.0	8.1	5.2	77.3
Mar. 6	0.8	40	437	51	7.5	116	47.6	38.7	81.4	3.0	3.7	15.6	19.2	0.5	2.1	31.1	14.2	7.1	4.7	75.9
Mar. 8	0.8	39	391	46	7.5	116	41.3	34.4	83.3	3.5	4.2	13.2	15.8	0.5	3.8	27.8	12.9	6.8	4.7	74.8
Mar.13	0.8	37	464	53	7.8	114	47.2	38.5	81.6	4.1	5.0	14.3	17.5	0.6	4.7	23.7	11.8	6.4	4.1	72.7
Mar.15	0.8	40	387	49	6.9	115	45.6	37.6	82.5	3.3	4.0	14.2	17.2	0.4	4.0					76.8
Mar.21	0.8	40~43	465	57	7.2	115	43.0	34.5	80.2	3.0	3.7	16.8	20.9	0.4	2.8	29.8	12.5	5.4	3.7	60.5
Mar.23	0.8	44	419	56	6.5	114	48.2	40.1	83.2	3.4	4.1	13.4	16.1	0.5	2.5	27.8	13.2	6.4	4.4	71.6
Apr. 2	0.8	39~42	365	46	7.0	115	45.9	38.1	83.1	3.2	3.9	13.7	16.5	0.5	5.5	36.2	17.3	10.2	6.4	82.8
Apr.10	0.8	40	606	77	6.9	115	72.9	57.9	75.2	3.9	4.9	16.7	21.0	0.5	3.1	14.6	9.1	5.1	3.1	75.2
Apr.12	0.8	38~42	646	81.6	6.5	115	78.3	62.2	79.4	4.2	5.3	16.4	20.7	0.7	4.5	15.5	8.2	4.8	3.3	72.2

Tenth United States/Japan Conference
on Sewage Treatment Technology

Centralized Sludge Treatment in Yokohama

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Sewage Works Bureau, City of Yokohama

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1. Introduction

Eleven sewage treatment plants have come into operation according to plan over the past two decades. As of March 1985, the population served by the Sewer System was 1.78 million, or 60 percent of the total population of Yokohama. Population growth has rapidly increased the volume of sludge requiring treatment. In the past, sludge was treated at the sewage treatment plants where it was produced. But this separate treatment method has become inadequate in view of the increasing quantities of sludge to be treated and disposed of, and in consideration of questions of environmental impact. New responses have become necessary.

To improve sludge treatment efficiency, we have drawn up and begun putting into execution a plan to collect the sludge from all of the sewage treatment plants and treat it at special sludge treatment centers in the Nanbu (southern) and Hokubu (northern) coastal districts. This report describes the planning and execution of collective treatment by our Bureau.

2. Yokohama's Sewer System and System Planning

One of Japan's foremost commercial and industrial cities, Yokohama boasts an international port with Japan's largest trade volume and participation in Japan's largest industrial region, the Keihin Industrial Zone. It also serves as a bedroom community for nearby Tokyo, Japan's capital and largest city. These factors have contributed to making Yokohama the second largest city in Japan. It has a population of about 2.9 million and covers an area of some 430 square kilometers.

As Figure 1 shows, the city's population and territory have undergone major changes over the years. Although the population has grown practically without interruption since Yokohama was officially designated a city in 1889, postwar growth has been especially spectacular.

In the course of rapid urbanization, comprehensive planning and improvement of such basic segments of the infrastructure as roads and sewers were disrupted first by the Great Kanto Earthquake of 1923, then by the bombing in World War II and expropriation of land by the American occupation forces. Their presence in Yokohama delayed the installation of sewer systems as compared to other major cities. The effects of this delay are still felt.

Sewer construction in Yokohama began in 1870 with the laying of sewers in the foreign concession, located in what is now Yamashita-cho in Naka Ward. Sewers were later laid in some parts of the downtown area, but there was very little systematic improvement and expansion of the system before the war.

Systematic improvement began in 1950 when a flood control project was launched in parts of Tsurumi Ward in northeast Yokohama. Expansion of the system and construction of sewage treatment plants was initiated in 1957 with improvements in the Chubu treatment district.

In response to rapid urbanization, five-year improvement plans were formulated beginning in 1963. From the first five-year plan (1963-1967) to the fifth (1981-1985), a total of approximately ¥1,375.9 billion was invested to expand the sewer system quickly. The second five-year plan (1968-1973) was revised in 1969 to effect expansion of the system to cover the entire city.

Under the revised second plan, the city's 430 square kilometers were divided into nine sewage treatment districts according to topographical and other features. One sewage treatment plant was constructed in each of these except the Hokubu and Totsuka districts where two were built. Almost all these sewage treatment plants were equipped to treat both sewage and sludge.

The city had two types of sewer systems. The coastal areas, which had been intensively developed for decades, were provided with combined sewer systems. The inland areas, on the other hand, to which rapid urbanization had come with the period of great economic growth in the mid-1960s, had separate systems.

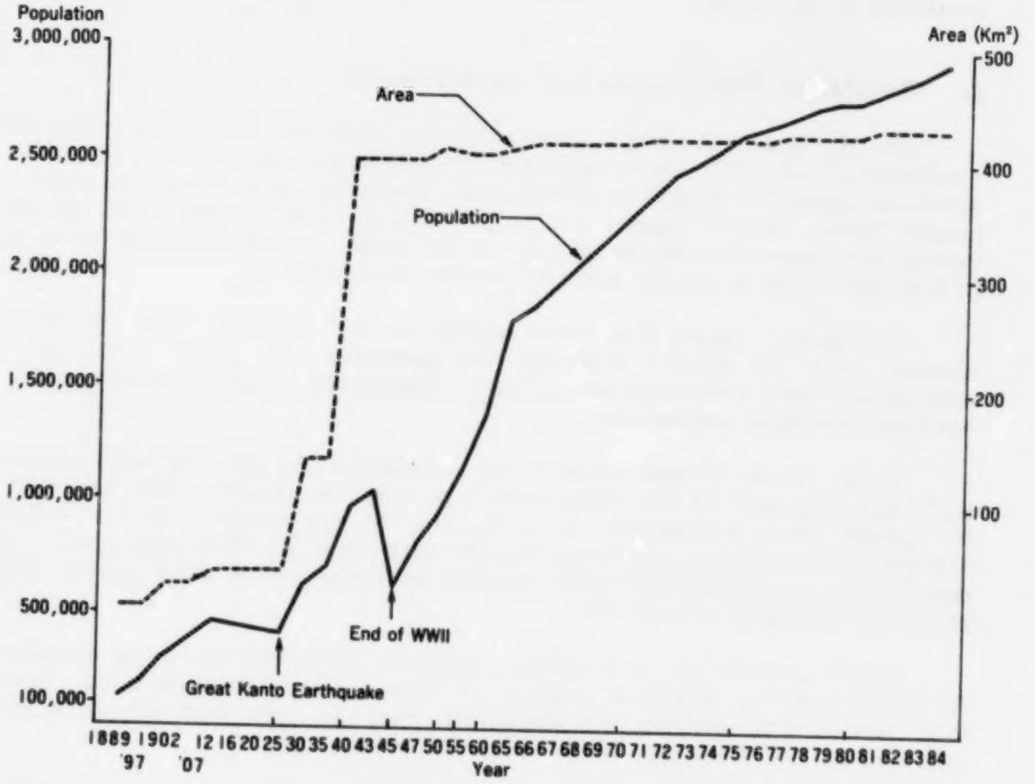


Figure 1. Trends in Yokohama's population and area under municipal jurisdiction

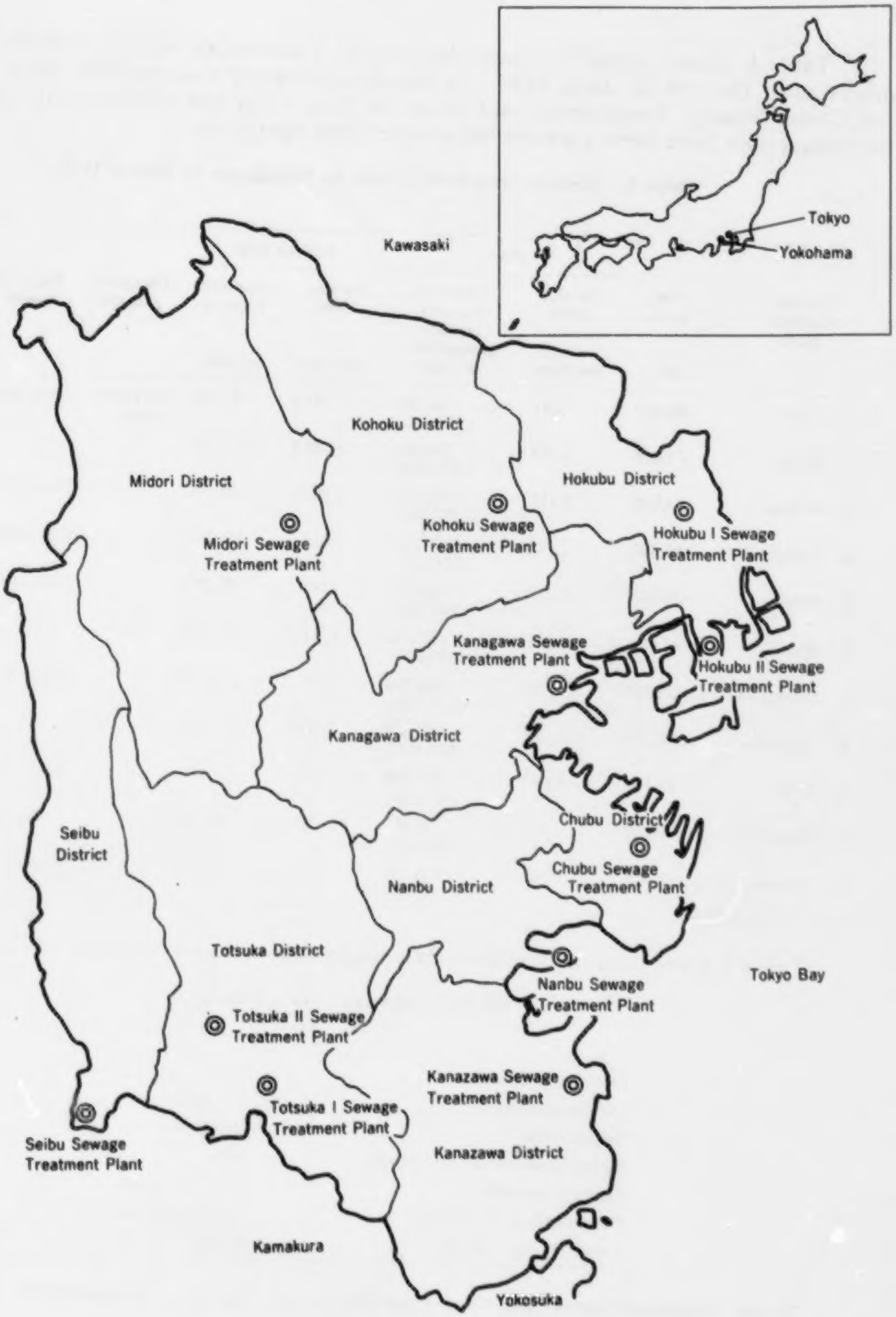

Tokyo
Yokohama
Kawasaki
Kohoku District
Midori District
Hokubu District
Kohoku Sewage Treatment Plant
Hokubu I Sewage Treatment Plant
Midori Sewage Treatment Plant
Kanagawa Sewage Treatment Plant
Hokubu II Sewage Treatment Plant
Kanagawa District
Chubu District
Seibu District
Chubu Sewage Treatment Plant
Nanbu District
Totsuka District
Tokyo Bay
Nanbu Sewage Treatment Plant
Totsuka II Sewage Treatment Plant
Kanazawa Sewage Treatment Plant
Totsuka I Sewage Treatment Plant
Seibu Sewage Treatment Plant
Kanazawa District
Kamakura
Yokosuka

Figure 2. Sewage treatment districts and plants in Yokohama

Table 1 shows capacity usage levels for Yokohama's eleven sewage treatment plants as of the end of fiscal 1984. In the approximately two decades since 1962 when the Chubu Sewage Treatment Plant came on line, all of the eleven plants called for in the initial plan have been constructed and put into operation.

Table 1. Sewage treatment plants in Yokohama in March 1985

		Plan		Service Area				
Sewage treatment plant	Site area m²	Service area hectares	Treatment capacity (service population) m³/day	Service area hectares	Treatment capacity m³/day	Treatment process	Receiving water	Month operation initiated
1. Chubu	68,300	1,287	96,300 (185,000)	781.9	47,000	Activated sludge	Tokyo Bay	Apr. 1962
2. Nanbu	70,620	2,458	225,000 (433,000)	2,038.5	225,000	〃	〃	Jul. 1968
3. Hokubu I	84,520	2,150	196,000 (377,000)	1,993.4	196,000	〃	Tsurumi River	Jul. 1968
4. Totsuka II	88,450	4,232	206,000 (396,000)	1,655.2	131,000	〃	Kashio River	Oct. 1972
5. Kohoku	125,360	6,270	439,000 (844,000)	1,245.2	67,800	〃	Tsurumi River	Dec. 1972
6. Midori	87,000	8,096	433,000 (833,000)	1,566.8	108,300	〃	〃	May 1977
7. Kanagawa	103,000	5,049	543,200 (1,074,000)	2,641.3	272,000	〃	Tokyo Bay	Mar. 1978
8. Kanazawa	236,070	5,082	345,000 (663,000)	2,480.2	172,500	〃	〃	Oct. 1979
9. Seibu	83,470	4,087	191,000 (367,000)	172.4	31,800	〃	Sakai River	Mar. 1983
10. Hokubu II	370,000	974	86,400 (166,000)	208.5	43,200	〃	Tokyo Bay	Aug. 1984
11. Totsuka I	31,260	2,004	124,000 (238,000)	675.7	31,000	〃	Itachi River	Dec. 1984

Table 2 gives data on Yokohama's sewage network as of the end of fiscal 1984.

Table 2. Sewage network data

Flush toilet service rate	60%
Service population	1.78 million
Service area	15,460 ha
Service area as percent of total	36%
Plants in operation	11 (11 designed)
Pump stations in operation	16 (25 designed)
Length of pipe network	5,726km

Sewer improvements are near completion in the old, long-served coastal areas, and are still advancing in peripheral inland areas.

3. Sludge Treatment and Disposal Today

The methods of sludge treatment and disposal employed at the different sewage treatment plants as of the end of fiscal 1984 are shown in Table 3.

Table 3. Sludge treatment and disposal (March 1985)

Sewage treatment plant	Average volume of sewage treated (m³/day)	Sludge treatment		Dewatered cake production (metric tons/year)
		Treatment process	Method of disposal	
1. Chubu	68,600	Anaerobic sludge digestion, followed by vacuum dewatering	Incineration, landfill	5,846
2. Nanbu	199,000	Anaerobic sludge digestion, followed by centrifugal dewatering	Incineration, landfill, drying	26,565
3. Hokubu I	138,000	Pipe to Hokubu II Sewage Treatment Plant	—	
4. Totsuka II	75,000	Thickened sludge is dewatering directly	Incineration, landfill	21,697
5. Kohoku	61,000	Anaerobic sludge digestion, followed by pressure dewatering	Landfill	8,268
6. Midori	38,700	Thickened sludge trucked to Kohoku Sewage Treatment Plant		
7. Kanagawa	187,100	Thickened sludge is dewatering directly	Incineration, landfill	42,137
8. Kanazawa	95,200	Piped to Nanbu Sludge Treatment Center	Incineration, landfill	3,935
9. Seibu	3,530	Thickened sludge trucked to Nanbu Sewage Treatment Plant		
10. Hokubu	16,800	Piped to Hokubu Sludge Treatment Center	Incineration, landfill	31,846
11. Totsuka I	9,340	Piped to Totsuka II Sewage Treatment Plant		

Generally speaking, there are three types of sludge treatment: i) gravity thickening followed by digestion and dewatering; ii) gravity thickening followed by wet air oxidation and dewatering; and iii) gravity thickening followed by dewatering.

The dewatered cake produced by wet air oxidation is disposed of in landfills, whereas most of the dewatered cake produced by digestion and dewatering and undigested dewatered cake is incinerated before landfill disposal.

The incinerators, as explained below, are installed at the sludge treatment centers, and the dewatered cake is trucked in from the various sewage treatment plants where it is produced. Figures 3 through 6 show total volumes and volumes for the various types of thickened sludge over the past ten years.

During the past decade, the flush toilet service rate has jumped from 27 percent to 60 percent. Along with this rapid increase, the amounts of both thickened sludge and dewatered cake have trebled.

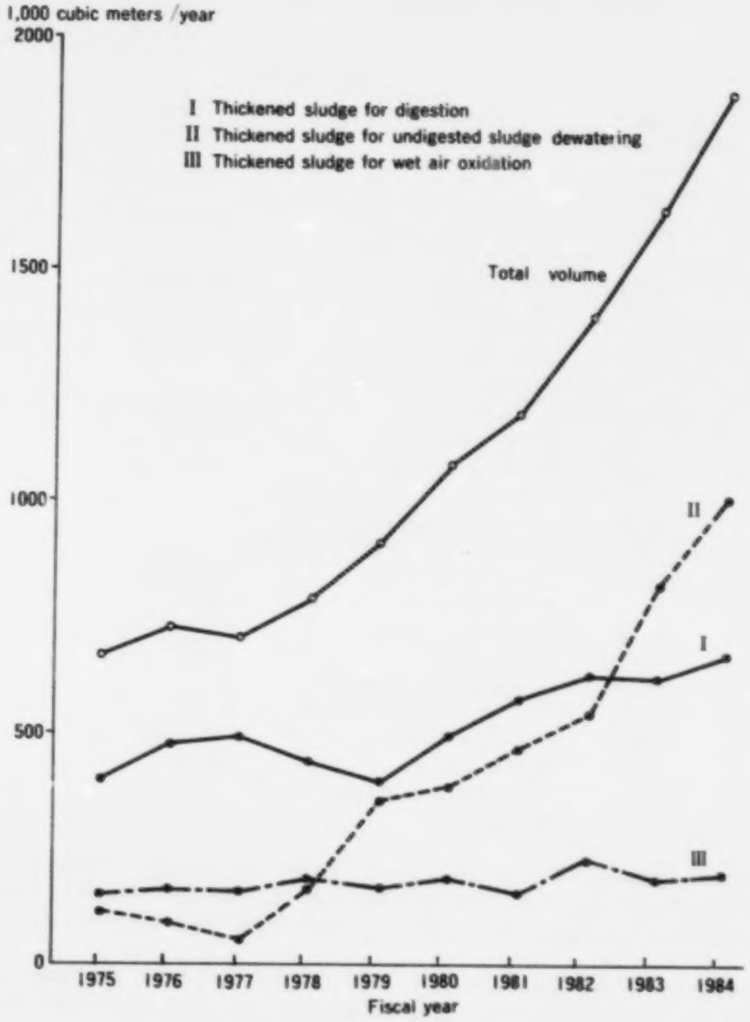


Figure 3. Trends in annual thickened sludge production

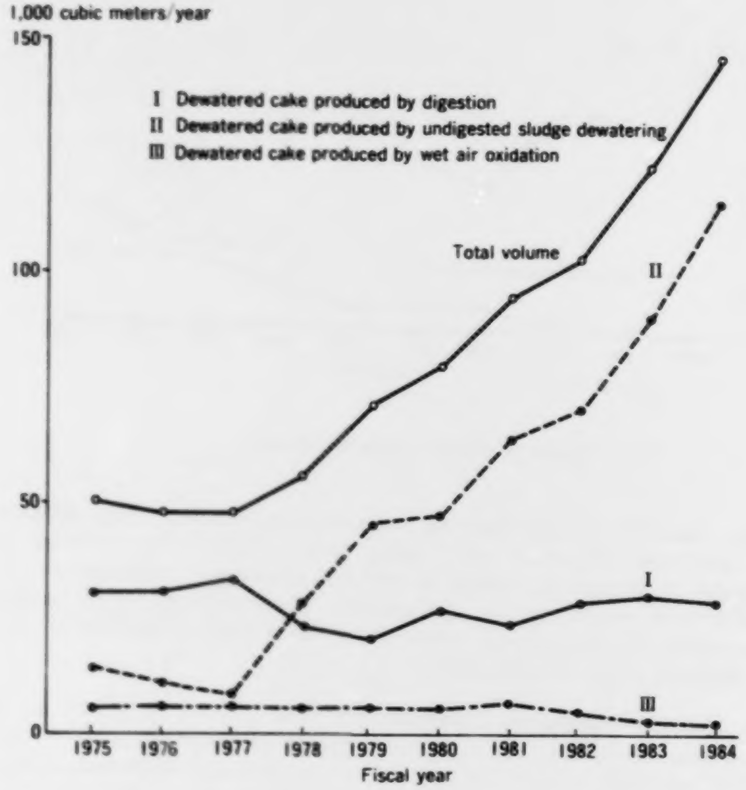


Figure 4. Trends in annual dewatered cake production

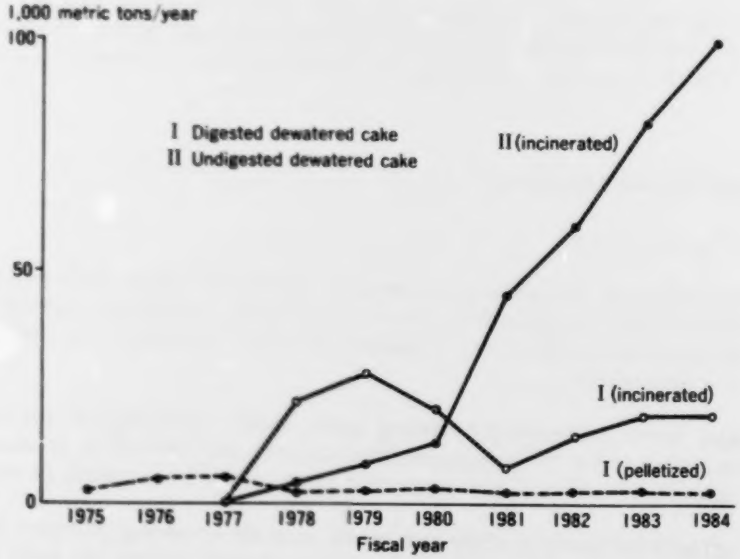


Figure 5. Trends in incineration and pelletization

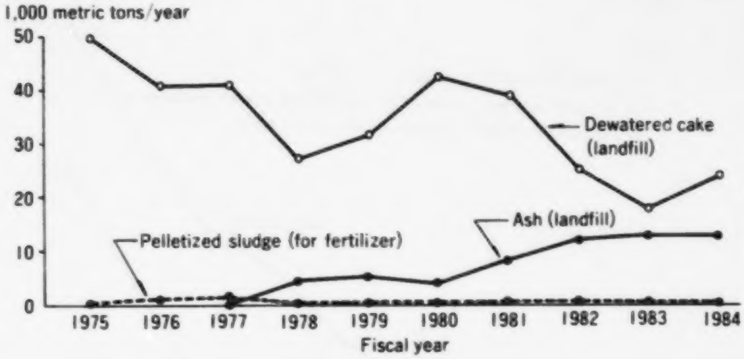


Figure 6. Sludge disposal trends

Yokohama's landfill sludge disposal became inadequate. The sudden growth in sludge volume and the difficulty of finding new landfill sites within the city led Yokohama to begin incinerating sludge to reduce disposable volumes in 1977. Sludge incineration permits a reduction in the amount of dewatered cake to between one-seventh and one-tenth of its original volume. This is very effective in extending the length of time disposal sites can be used.

Sludge disposal in Yokohama continues to have a landfill policy as its core, but because sludge is also considered an important resource, steps have been taken to use it effectively. One method in use since 1977 is to turn the digested sludge into dried pellets and to use the pellets as fertilizer at such places as golf courses, parks and farms.

Ash from the incinerator contains unslaked lime which hardens on contact with moisture and can therefore be used as a construction material in roadbed and other such construction. The city is now studying the technical and economic aspects of these uses.

4. Centralized Sludge Treatment

4.1 Sludge collection systems

Yokohama designed its sewage treatment plants to treat both waste water and sludge. The environment surrounding the city's sewage treatment plants has changed markedly in the years since land was acquired for their construction, however, because of its intensive urbanization.

There have been increasingly strong calls from residents of surrounding communities for measures to be taken against the malodor and the noise produced by sludge treatment facilities. These calls are expected to continue to increase in volume.

Present efforts to control offensive odors include covering primary sedimentation tanks and sludge thickeners at all the sewage treatment plants, as well as restricting the hours during which sludge treatment facilities can be operated.

Thus, from the viewpoints of both facility maintenance and community pressure, circumstances increasingly favor avoiding wherever possible further expansion of individual sludge treatment facilities at existing sewage treatment plants.

In response to this situation, we have developed plans to build centralized sludge treatment centers where increasingly massive quantities of sludge can be treated without creating environmental problems. Centralizing sludge treatment is expected to have the following effects:

a. Increase sludge treatment efficiency in terms of treatment facility construction and maintenance.

b. Reduce the amount of pollution control equipment required at sewage treatment plants.

c. Improve the residential environment in the area surrounding sewage treatment plants and reduce the congestion and inconvenience caused by truck transportation of dewatered cakes by piping sludge from the plants to the sludge treatment centers.

d. Eliminate the need to treat recycled process water from sludge treatment equipment at sewage treatment plants, improving secondary effluent quality.

e. Centralize and increase efficiency of process water treatment at the sludge treatment centers.

f. Allow comprehensive use and recovery of energy from sludge by making greater volumes available through sludge collection and centralized treatment.

With these goals in mind, it was decided to build sludge treatment centers at the Hokubu II and Kanazawa sewage treatment plants, where sufficient land had been acquired beforehand, and to begin centralized treatment of sludge. The construction plan involved dividing the eleven sewage treatment plants into two groups as shown in Figure 7. Sludge from the Midori, Kohoku, Hokubu I, Hokubu II, and Kanagawa sewage treatment plants would be piped to the Hokubu Sludge Treatment Center, while sludge from the Chubu, Nanbu, Kanazawa, Totsuka I, Totsuka II, and Seibu plants would be piped to the Nanbu Sludge Treatment Center for centralized treatment.

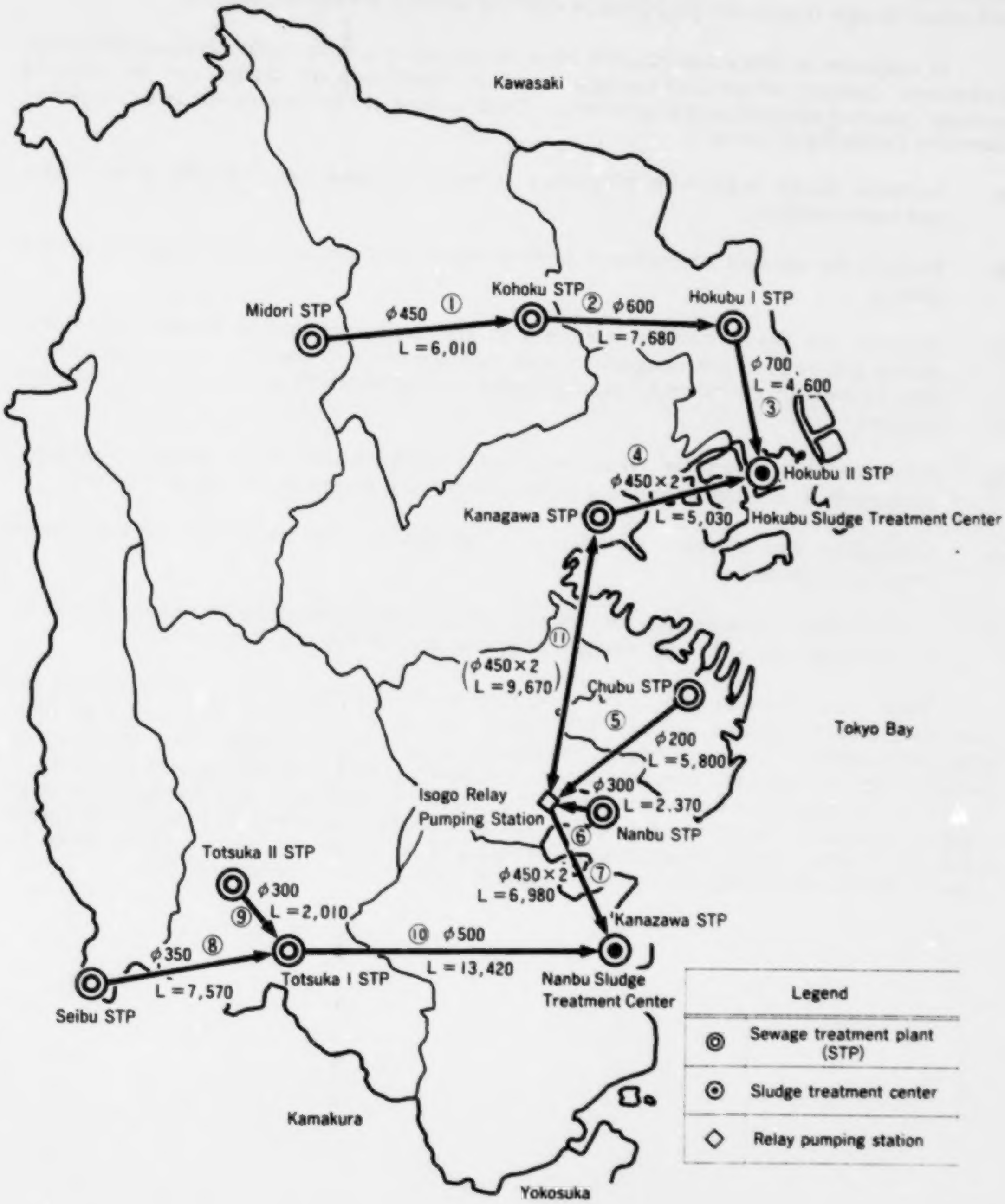

Kawasaki
Midori STP
φ450 ①
L = 6,010
Kohoku STP
② φ600
L = 7,680
Hokubu I STP
φ700
L = 4,600
③
④
φ450 × 2
L = 5,030
Hokubu II STP
Hokubu Sludge Treatment Center
Kanagawa STP
⑪
φ450 × 2
L = 9,670
Chubu STP
⑤
φ200
L = 5,800
Tokyo Bay
Isogo Relay
Pumping Station
φ300
L = 2,370
⑥
Nanbu STP
φ450 × 2
L = 6,980
⑦
Kanazawa STP
Totsuka II STP
φ300
⑨
L = 2,010
⑩ φ500
L = 13,420
φ350 ⑧
L = 7,570
Seibu STP
Totsuka I STP
Nanbu Sludge
Treatment Center
Legend
◎ Sewage treatment plant (STP)
⊙ Sludge treatment center
◇ Relay pumping station
Kamakura
Yokosuka

Figure 7. Map of planned sludge pipe network

The pipelines do not link the various sewage treatment plants directly to the sludge treatment center. Instead sludge is sent from one plant to the next in the direction of the treatment center. Table 4 supplies details of the pipeline network.

Table 4. Sludge pipeline data

Data Route No.	Diameter (mm) × no. of pipes	Length(m)	Remarks
1.	450 × 1	6,010	Completed
2.	600 × 1	7,680	Under construction
3	300 × 1 700 × 1	4,600	φ300 completed Under construction
4.	450 × 2	5,030	Under construction
5.	200 × 1	5,800	Completed
6.	300 × 1	2,370	〃
7.	450 × 2	6,980	Under construction
8.	350 × 1	7,570	〃
9.	300 × 1	2,0[illegible]0	Not yet began
10.	500 × 1	13,420	〃
11.	450 × 2	9,670	Under construction

In view of a future interchange between the two groups, a route was planned linking the Kanagawa Sewage Treatment Plant with the Isogo Relay Pumping Station. Part of this line has already been completed.

Table 5 provides data on the two sludge treatment center facilities.

Table 5. Sludge treatment center data

*Sludge containing 99% water

Sludge treatment center	Sewage treatment plants served	Volume of sludge received *	Sludge treatment process
Hokubu	Midori	m^3/day	Thickening (by gravity & centrifugation) ↓
	Kohoku		Anaerobic digestion ↓
	Hokubu I	45,000	Dewatering (by centrifugation) ↓ ↘
	Hokubu II		
	Kanagawa		Incineration, Pelletization for fertilizer
			Thickening (by gravity & centrifugation) ↓
			Wet air oxidation ↓
			Dewatering
Nanbu	Chubu	m^3/day	Thickening (by centrifugation) ↓
	Nanbu		Anaerobic digestion ↓
	Kanazawa	31,000	Dewatering (by pressurization) ↓
	Totsuka I		Incineration
	Totsuka II		
	Seibu		Thickening (by centrifugation) ↓
			Wet air oxidation ↓
			Dewatering (by pressurization)

Three types of sludge treatment are planned for the Hokubu Sludge Treatment Center. These involve combinations of the thickening, anaerobic digestion, wet air oxidation, dewatering, pelletization (into fertilizer) and incineration processes.

Two types of sludge treatment are planned at the Nanbu Sludge Treatment Center, involving combinations of the thickening, anaerobic digestion, wet air oxidation, dewatering, and incineration processes. The plurality of processes at both centers is intended to increase sludge treatment flexibility and facilitate responses to future technological innovations. Construction of the centers is proceeding in stages, allowing for maximum use of existing sludge treatment equipment at sewage treatment plants.

The pipeline will consist of single or double ductile cast iron pipes ranging from 200 to 700 mm in diameter with a total length of about 88 kilometers. To date only single-pipe lines are in place, but a two-lane network is planned for the future to facilitate response to any damage or trouble in the pipeline.

Specifications for the pipeline are as follows:

a. Piped sludge: a mixture of raw sludge and excess sludge at a concentration of TS 1 percent.

b. Flow velocity: 1-1.5 m/sec.

c. Pipe diameter: sufficient to allow completion of sludge piping in 20 hours of daily operation, in view of the need for pipe washing and maintenance.

d. Equipment: gate valves, air valves, blow-off valves, and others, as required.

Facilities include sludge storage tanks, sludge pumps, and sludge receiving tanks. Existing thickening tanks and digestion tanks have been rebuilt for use as sludge storage tanks; new sludge receiving tanks have been built.

One of the main items of sludge piping equipment is the sludge pump. A non-clog centrifugal pump was chosen because of its economical efficiency and its ability to provide the reliability required for transporting sludge over great distances.

The operating organization of the feeding and receiving facilities was designed with the need for a continuous system linking the sludge treatment facilities to the piping facilities in mind. Control of the sludge feed system operation is centralized at the sludge treatment centers, from which commands are sent to the sludge feeding equipment at the different sewage treatment plants.

4.2 Construction status

The centralized sludge treatment system is still under construction. As of the end of fiscal 1984, construction had begun on most of the sludge pipe routes. Sludge was already being piped on Route 1 from the Midori to the Kohoku sewage treatment plants, and on Route 3 from the Hokubu I to the Hokubu II plants. Almost all of the routes are expected to be in operation by fiscal 1988.

Table 6 shows the state of progress in construction of sludge treatment equipment at the centers.

Table 6. Status of sludge treatment center facilities

	Facility	Capacity	Quantity	Remarks
Hokubu Sludge Treatment Center	Egg-shaped digestion tank	6,800m^3/tank	12 tanks	Under construction
	Thickening tank	1,200m^3/tank	8 tanks	
	Centrifugal dewaterer	50m^3/hour	3	
	Incinerator (fluidized bed)	100t/day	2	
Nanbu Sludge Treatment Center	Wet air oxidizer	571m^3/day	2 side-by-side pipes	20 metric tons of dry solids/day
	Incinerator (vertical-multistage)	100t/day	1	
	Incinerator (fluidized bed)	150t/day	1	Drier

Construction of the incinerators was given priority over other processing equipment scheduled to be introduced at the sludge treatment centers. The dewatered sludge cakes produced at the sewage treatment plants are now brought to the center by truck for incineration. Incineration reduces their volume and extends landfill site use.

5. Overview of Sludge Treatment Centers

5.1 Unit processes

The Hokubu and Nanbu Sludge Treatment Centers employ combinations of the unit processes that are currently thought most likely to ensure maximum sludge treatment efficiency. The following explains why these processes were chosen and describes them in simple terms.

5.1.1 Thickening

The gravity thickening method was used in Japan in the past, but because the organic content of sludge has risen sharply in recent years and sludge putrefaction has made it impossible to obtain the desired concentrations, mechanical thickening is now usually employed.

Mechanical thickening is used to thicken only excess sludge, while gravity thickening is used to thicken only raw sludge. For the following reasons, it is standard procedure at both of our sludge treatment centers to mechanically thicken all sludge:

a. In a pumped sludge system, it is difficult to separate and pipe both excess and raw sludge.

b. Sludge putrefaction is difficult to prevent when the sludge must be piped over long distances.

There are two main types of mechanical thickening, flotation and centrifugation. Experience with both has accumulated since 1975. Mechanical thickening makes it possible to maintain higher and relatively more stable concentrations of solids in the concentrated sludge than does gravity thickening.

With centrifugation, electricity and maintenance costs rise as the scale of operation increases, but operating procedures are simpler and less manpower is required than with flotation.

Flotation uses less electricity but requires a larger site than centrifugation. In addition, plant construction costs are higher, the variety of auxiliary equipment required makes maintenance difficult, vastly greater quantities of tank gas must be deodorized and environmental protection costs increase.

After studying the above factors, Yokohama decided to adopt centrifugation thickening.

5.1.2 Anaerobic digestion

Sludge digestion is a stable method of treatment which is commonly used in Japan and which has given good results at the Chubu, Nanbu, and Kohoku sewage treatment plants in Yokohama. The anaerobic digestion process has been adopted for Yokohama's sludge treatment centers because it allows for energy recovery in the efficient form of methane gas.

Digestion tanks can be classified according to shape as Anglo-Saxon, classic continental, or egg-shaped tanks. Although Yokohama had used cylindrical tanks in the past, it became the first city in Japan to opt for egg-shaped digestion tanks in consideration of the following advantages with respect to other types of tank:

a. They require less energy to stir and include little or no internal dead space.
b. They are structurally efficient, are not subject to stress concentrations and can be made very large.
c. They are not prone to sand sedimentation because of their shape or for other reasons.
d. They produce less scum.
e. They release less heat.
f. Their shape is attractive.

5.1.3 Wet air oxidation

Wet air oxidation of sludge is a method of combusting organic substances in sludge in the presence of water using oxygen from the air. It is characterized by:

a. Simultaneous digestion and incineration of sludge in a single process;
b. Easy dewatering of ashes without chemical conditioning; and
c. Compact equipment and modest space requirements.

A wet air oxidizer with a solids capacity of 20 metric tons a day has been in operation at the Hokubu I Sewage Treatment Plant since 1968. It was the first to be put into regular use in Japan. A second oxidizer with the same capacity was installed in 1973.

The performance of these two machines has proved the method's excellence. It is technically stable and safe in terms of both process and equipment and is relatively cost efficient. It was consequently selected as one of the treatment processes at the sludge treatment centers.

5.1.4 Dewatering

Types of dewatering equipment currently in use in Japan include vacuum filters, pressure filters, centrifuges and belt press filters. Of these, all but belt press filters have been used recently in Yokohama. The dewatering devices employed at the sludge treatment centers were chosen to meet the following requirements:

a. A high sludge treatment capacity necessitated by the massive volumes of sludge collected from the different sewage treatment plants;
b. A low manpower requirement to derive maximum benefit from the centralization of sludge treatment;
c. Production of low-water-content dewatered cake to facilitate incineration; and
d. Maximum recovery of solids.

It was decided on the basis of practical results to employ a centrifuge at the Hokubu Sludge Treatment Center and a high-pressure belt-press filter at the Nanbu Sludge Treatment Center.

5.1.5 Pelletization (as fertilizer)

At the Nanbu Sewage Treatment Plant, about 2,800 metric tons a year of dewatered cake with a polymer coagulant added after anaerobic digestion are currently being dried and pelletized using a rotary drier and sold as a fertilizer to golf courses and parks.

The utility of sanitary sludge fertilizer is well known. Sludge recycling to agricultural and park land is not only beneficial to the soil but an excellent method of sludge disposal. For sludge to be easily used as a fertilizer, however, its water content must be lowered considerably. Driers serve this purpose.

Plans are for about one-third of the sludge treated at the Hokubu Sludge Treatment Center to be dried for conversion to fertilizer.

5.1.6 Incineration

Sludge incineration in Japan began in the early 1960s as a measure to reduce sludge volumes. The various types of sludge incinerators include flash-drier incinerators, vertical-multistage incinerators, rotary-drier incinerators and fluidized-bed incinerators. The type most widely used today is the vertical-multistage incinerator, followed by the fluidized-bed incinerator. It was decided to use these two tried-and-tested types at the Hokubu and Nanbu sludge treatment centers.

The fluidized-bed incinerator produces exhaust gases with temperatures in the area of 800°C. An economical and energy-saving incinerator has been developed by installing a drier at the previous stage which uses the incinerator's hot gases to dry dewatered cake before incineration.

This completes our description of the unit processes now in use in Yokohama. Sludge treatment requires the full exploitation of the features of each process as well as an intelligent combination of the processes themselves.

Figure 8 is a flow sheet showing the processes determined to be most effective at Yokohama's sludge treatment centers.

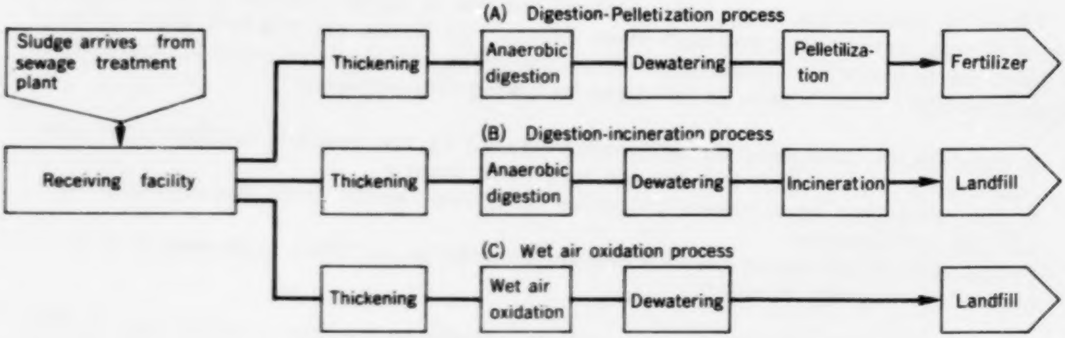

(A) Digestion-Pelletization process
Sludge arrives from sewage treatment plant
Receiving facility
Thickening
Anaerobic digestion
Dewatering
Pelletiliza-tion
Fertilizer
(B) Digestion-incineration process
Thickening
Anaerobic digestion
Dewatering
Incineration
Landfill
(C) Wet air oxidation process
Thickening
Wet air oxidation
Dewatering
Landfill

Figure 8. Sludge flow at the sludge treatment center

5.2 Present and planned facilities

5.2.1 The Hokubu Sludge Treatment Center

a. Present facilities

The Hokubu Sludge Treatment Center is Yokohama's largest, treating about 60 percent of the sludge produced in the city. Four sewage treatment plants--Midori, Kohoku, Hokubu I and Kanagawa--send their sludge to the center. This sludge combined with that produced at the Hokubu II Sewage Treatment Plant adjacent to the center will amount, according to the master plan, to 45,600 m^3/day (with a 99 percent water content).

Facilities built to date comprise a waste water facility and two fluidized-bed incinerators burning powdered coal.

The segments of the planned sludge pipeline network already in operation are Route 1 from the Midori to the Kohoku plant and Route 3 linking the Hokubu I and Hokobu II plants. Other pipelines are now under construction. The Kohoku and Kanagawa plants are equipped with dewatering facilities. Until their pipelines are completed, they will continue to send dewatered cake to the Hokubu Sludge Treatment Center by truck. Sludge piped from the Hokubu I and produced at the adjacent Hokubu II plants is incinerated immediately following dewatering at the center.

b. Selection of treatment process

Table 7 shows treatment processes and volumes handled at the Hokubu Sludge Treatment Center.

Collected sludge is first thickened by gravity and centrifugation. Then one-third is subjected to anaerobic digestion, dewatering, and incineration; one-third to anaerobic digestion, dewatering, and drying; and the remaining third to wet air oxidation and dewatering.

The center is careful to recover energy from the collected sludge and to use it effectively. It emphasizes the anaerobic digestion-incineration process for this reason, and is constructing facilities to generate electricity from digestion gas and to use the waste heat from incinerators. These methods are essentially combinations of unit processes already in use for a long time, but they incorporate new technologies.

Table 7. Process flow at Hokubu Sludge Treatment Center

Treatment Process	Volume of treated solids (metric tons/day)	Thickening		Dewatering	Drying	Incineration	Wet air oxidation process	
		Gravity	Digestion Centrifugation					
A	151.9t	1,256m^3 ×8 tanks	100m^3/hr ×6 centrifuges	Anaerobic heating at 35°C for 30 days 6,800 m^3/tank× 12 tanks	Centrifugal dewatering 50m^3/hr× 4 centrifuges	Rotary kiln 48 metric tons/day× 8 kilns		
B	151.9t	1,256m^3× 8 tanks fuges	100m^3/hr× 6 centrifuges	Anaerobic heating at 35°C for 30 days 6,800m^3/tank 12 tanks	Centrifugal dewatering 50m^3/hr× 4 centrifuges		Fluidized-bed incinerator 20 metric ton/day× 1 kiln 30 metric tons/ day×3 kilns	
C	151.9t	1,256m^3× 8 tanks	100m^3/hr 6 centrifuges		Filter pressing 8.5kg/m^2/hr× 10 presses			Wet air oxidizer 20 metric tons/day× 8 oxidizers

The properties of sludge have changed in recent years, making it more difficult to thicken. Piped sludge is particularly prone to acidic fermentation. It becomes anaerobic after several hours. The resulting deterioration of its properties makes thickening especially difficult.

Recent poor results with gravity thickening require gravity thickened sludge to be further thickened by centrifugation. This lessens the amount of sludge sent to the digestion tanks and the amount of heat needed to warm it.

It has long been known that their shape enables the contents of egg-shaped tanks to be stirred thoroughly and that they afford highly efficient sludge digestion. They have not been used in the past, however, because of the high cost of installation and scaffolding and the underdeveloped state of prestressed concrete construction methods. The solution of these problems led to the construction of the first Japanese egg-shaped digestion tank. It has been in use at the Hokubu Sludge Treatment Center since 1983.

Design specifications for the center's egg-shaped tanks are as follows:

a. Design requirements: low-cost construction, small surface area, seismic structure for safety during earthquakes
b. Type: anaerobic two-stage digestion
c. Water content of thickened sludge: 95 percent
d. Water content of digested sludge: 95 percent
e. Digestion period: 30 days
f. Heating method: external heat exchanger, mesophilic digestion (at about 35°C)
g. Mixing method: mechanical and digestion-gas stirring
h. Digestion rate: 40 percent
i. Dimensions: inside diameter 21.8 m x 30.8 m deep; radius of curvature 15.55 m (egg-shaped)
j. Number of tanks: 24
k. Material grade: fissure-resistant, high-strength, water- and air-tight prestressed concrete
l. Methane gas utilization: power generation using digestion gas; incoming sludge heated by waste heat from gas engine

Figure 9 shows the layout of the Hokubu Sludge Treatment Center and Figure 10 the digestion and incineration flow sheet.

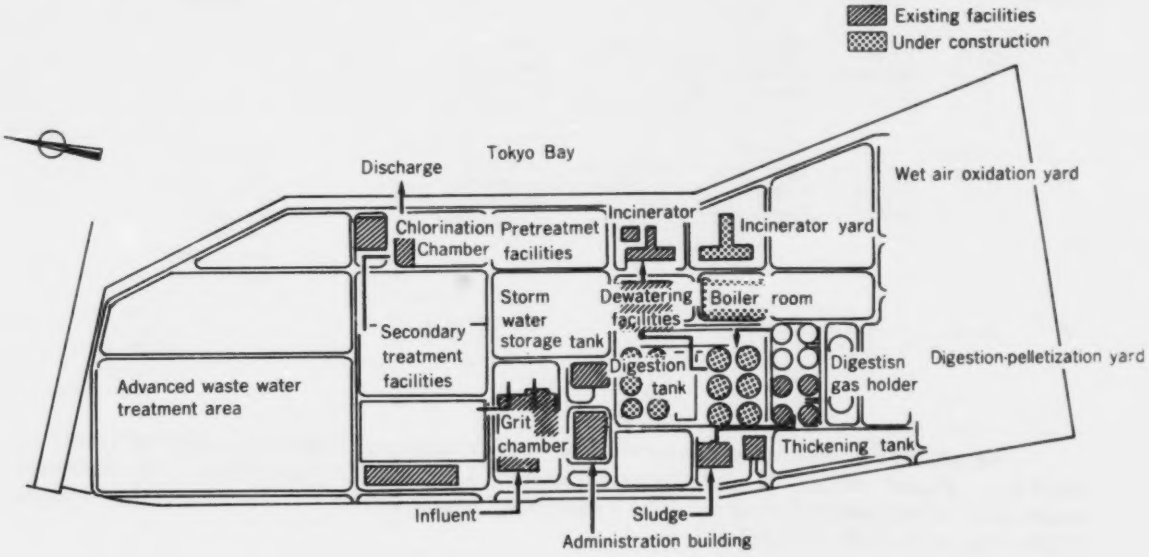

Existing facilities
Under construction
Tokyo Bay
Discharge
Wet air oxidation yard
Incinerator
Chlorination Chamber
Pretreatmet facilities
Incinerator yard
Storm water storage tank
Dewatering facilities
Boiler room
Secondary treatment facilities
Digestion tank
Digestisn gas holder
Digestion-pelletization yard
Advanced waste water treatment area
Grit chamber
Thickening tank
Influent
Sludge
Administration building

Figure 9. Layout of the Hokubu Sludge Treatment Center

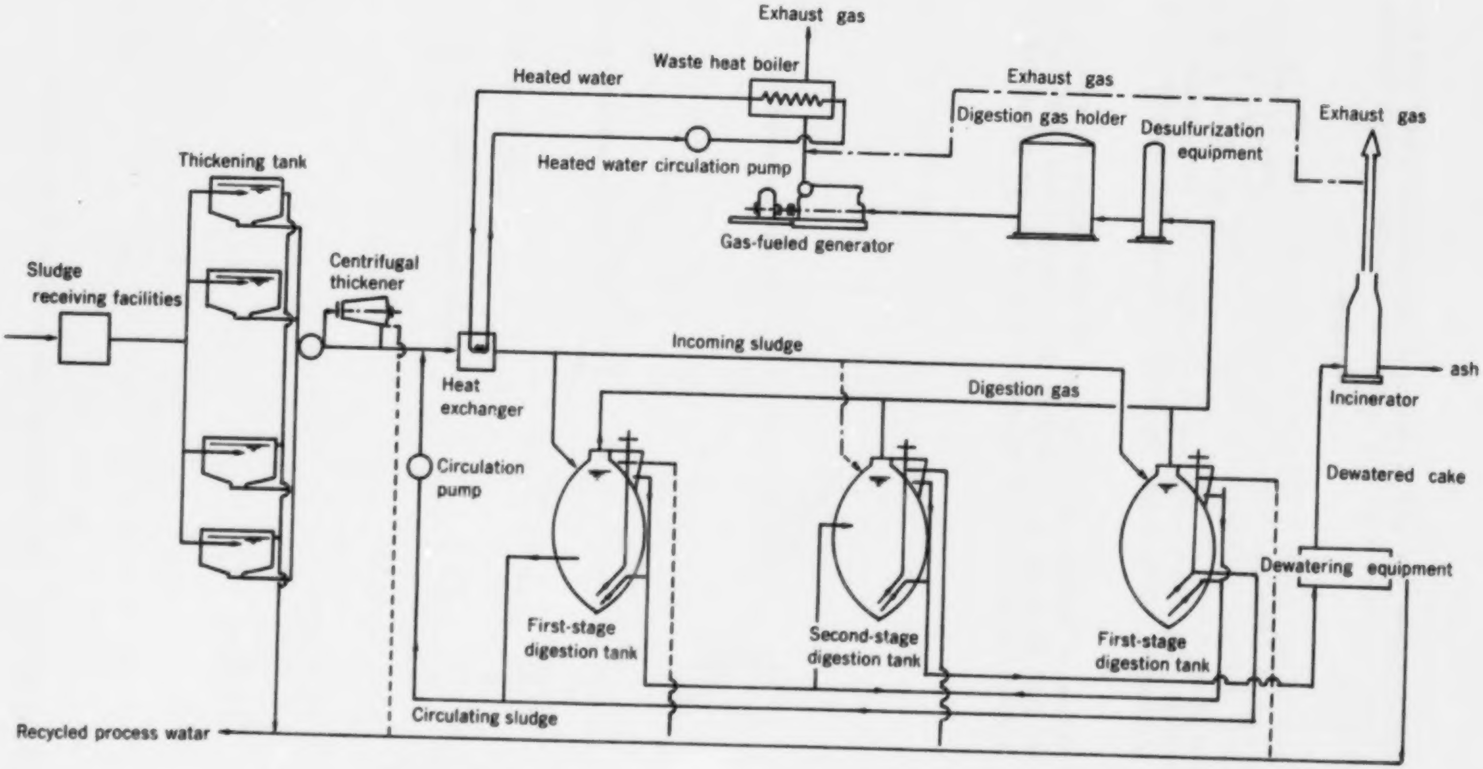

Exhaust gas
Waste heat boiler
Heated water
Exhaust gas
Digestion gas holder
Desulfurization equipment
Exhaust gas
Heated water circulation pump
Thickening tank
Gas-fueled generator
Sludge receiving facilities
Centrifugal thickener
Incoming sludge
ash
Heat exchanger
Digestion gas
Incinerator
Circulation pump
Dewatered cake
Dewatering equipment
First-stage digestion tank
Second-stage digestion tank
First-stage digestion tank
Circulating sludge
Recycled process watar

Figure 10. Digestion and incineration flow sheet

c. Construction plans

Figure 11 shows the construction plan and trends in sludge volumes treated at the center. Quantitative data were obtained from the growth in sewage amounts coming from the sewage treatment plants.

All the sludge produced at the Hokubu I Sewage Treatment Plant has been sent to the center since the summer of 1983 and treated by gravity thickening, centrifugal dewatering and incineration. In fiscal 1986 piping of sludge is to begin from the Midori, Kohoku, Hokubu I and Kanagawa sewage treatment plants, increasing the total volume of sludge treated to 95 metric tons of dry solids per day; sludge from these plants will be treated by gravity thickening, centrifugal thickening, anaerobic digestion and incineration.

Sludge volumes will increase linearly as the number of flush toilets increases, but for the time being priority will be given to expanding the digestion and pelletization processes. Centrifugal thickening and wet air oxidation processes are scheduled for adoption later.

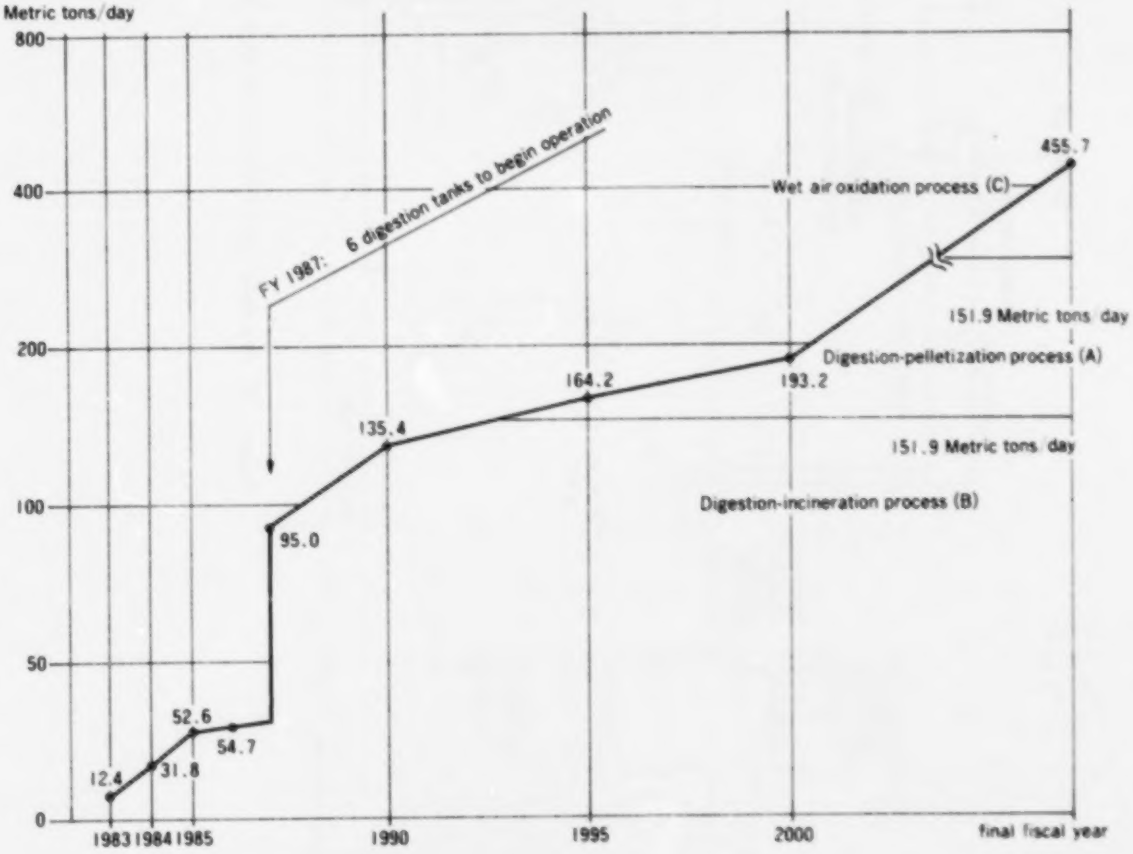


Figure 11. Construction plan at the Hokubu Sludge Treatment Center

5.2.2 Nanbu Sludge Treatment Center

a. Present facilities

The Nanbu Sludge Treatment Center is scheduled to receive a total of 31,000 m^3/day (with a 99 percent water content) of sludge produced at the Chubu, Nanbu, Totsuka I, Totsuka II, Seibu and Kanazawa sewage treatment plants.

Facilities completed thus far include two wet air oxidation lines with a capacity of 20 metric tons of dry solids per day each, one vertical multistage incinerator with a capacity of 100 metric tons/day and one drier-equipped fluidized-bed incinerator with a capacity of 150 metric tons/day. Because sludge pipes to the center are still under construction, dewatered cake produced at the sewage treatment plants is trucked to the center for incineration. The amounts involved are 5,850 metric tons/year from the Chubu plant, 21,750 metric tons/year from the Nanbu plant, 9,830 metric tons/year from Totsuka I and II plants and 1,000 metric tons/year from the Seibu plant. The 196,200 m^3/year of sludge produced at the adjacent Kanazawa Sewage Treatment Plant is treated by wet air oxidation.

Sludge treatment facilities are in operation at the Chubu, Nanbu, Totsuka II, and Seibu plants. Even after the sludge pipelines to the Nanbu Sludge Treatment Center begin operation, these plants will continue to treat their own sludge, in the interest of efficient use of existing facilities, and the excess will be piped to the Nanbu center. Equipment at the plants will not be replaced, however, so that the volumes they send to the center will gradually increase over time.

b. Treatment process selection

The processes used at large-scale collected sludge treatment facilities must allow for both the efficient treatment of large quantities of sludge and substantial economies of scale. At the same time, they must satisfy such requirements as adaptability to load fluctuations and back-up capability in the event of trouble elsewhere in the network.

The processes used in the system must also ensure flexibility in future sludge treatment. There is room for improvement in current sludge treatment and unit processes, and remarkable new sludge treatment technologies are still emerging.

Table 8 shows combinations of sludge treatment processes employed at the Nanbu Sludge Treatment Center.

Table 8. Process flow at the Nanbu Sludge Treatment Center

	Volume of treated solids (metric tons/day)	Thickening	Digestion	Wet air oxidation process	Dewatering	Incineration
B	95.0	Centrifugation: 100m^3/hr×6 centrifuges	Anaerobic heating at 35℃ for 30 days 6,400m^3/tank× 9 tanks		Belt press 150kg/m/hr× 3m×18 presses	Vertical multi-stage incinerator 100 metric tons/day× 1 incinerator fluidized-bed incinerator 150 metric tons/days×2 incinerators
B	35.3	Gravity: 522m^3×4 ponds		Moist oxidization 20 metric tons/day ×4 oxidizers 23 metric tons/day ×4 oxidizers	Pressure 14.5kg/m^2/hr ×6 presses	
	138.4	Centrifugation: 100m^3/hr ×8 centrifuges				

One-third of the expected volume of sewage is to be treated by centrifugal thickening, anaerobic digestion, belt-press dewatering and incineration (the combination known as the digestion-incineration process). The remaining two-thirds is to be treated by thickening, wet air oxidation and pressure dewatering. A gravity thickening tank will be used to concentrate 35.3 metric tons/day of the sludge produced at the Kanazawa Sewage Treatment Plant.

Figure 12 shows the layout of the Nanbu Sludge Treatment Center.

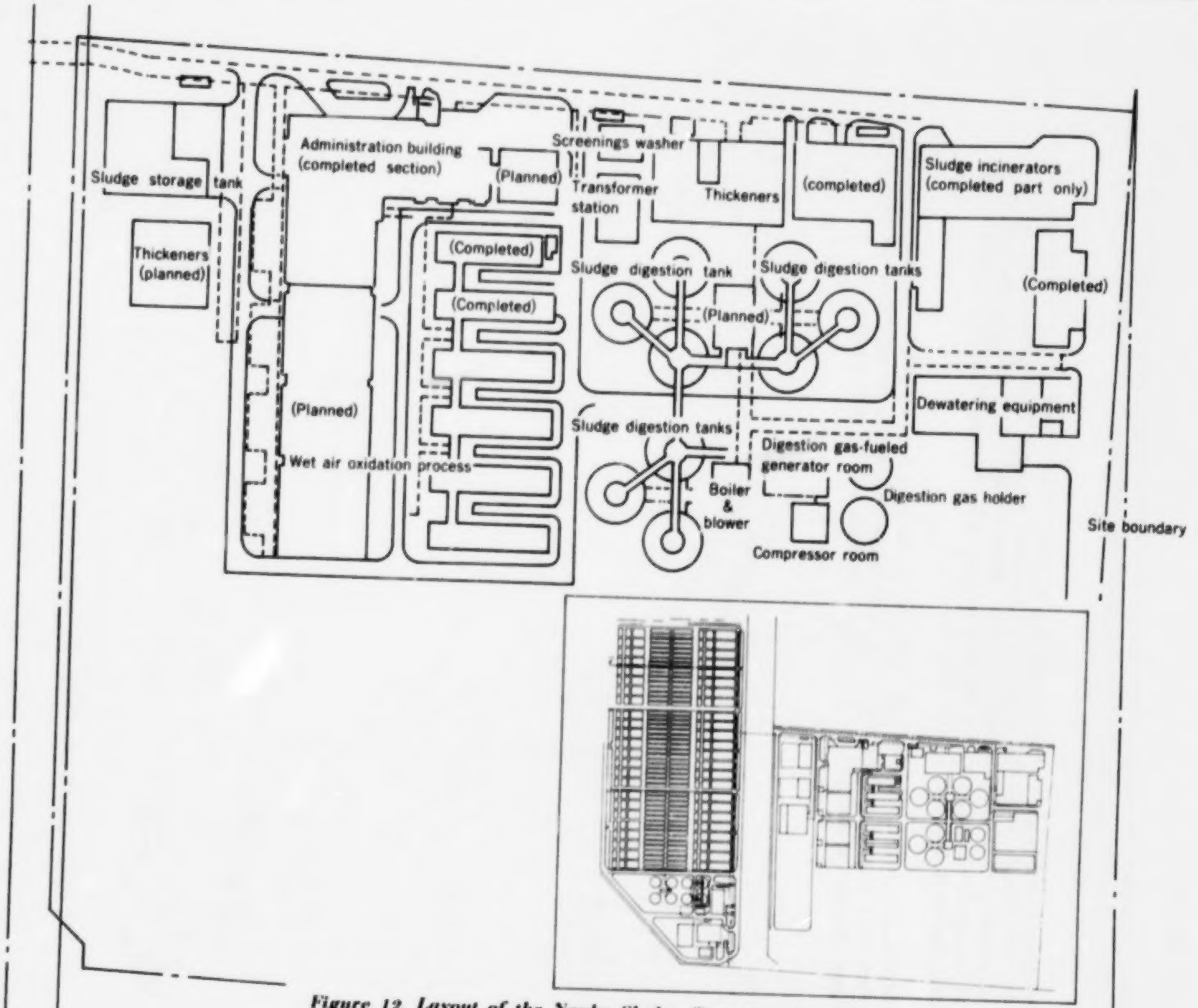

Administration building
(completed section)
(Planned)
Screenings washer
Transformer
station
Thickeners
(completed)
Sludge incinerators
(completed part only)
Sludge storage tank
Thickeners
(planned)
(Completed)
(Completed)
Sludge digestion tank
Sludge digestion tanks
(Planned)
(Completed)
(Planned)
Wet air oxidation process
Sludge digestion tanks
Digestion gas-fueled
generator room
Dewatering equipment
Boiler
&
blower
Digestion gas holder
Compressor room
Site boundary

Figure 12. Layout of the Nanbu Sludge Treatment Center

c. Construction plans

Figure 13 shows the construction plan for the center and trends in sludge volumes treated there. Quantitative data were obtained from the growth in amounts of sewage arriving from the sewage treatment plants.

The center will begin to receive sludge in 1988. Until this time, it is scheduled to treat only the sludge from the adjacent Kanazawa Sewage Treatment Plant using a gravity thickening, wet air oxidation process. When sludge piping begins, the volume of sludge will roughly double, and the existing process will be combined with a centrifugal thickening-digestion process not yet constructed.

Thereafter, sludge volumes will increase linearly as the number of flush toilets in the service area grows. Initial expansion will focus on the use of digestion processes. Centrifugal thickening and wet air oxidation processes will be added later.

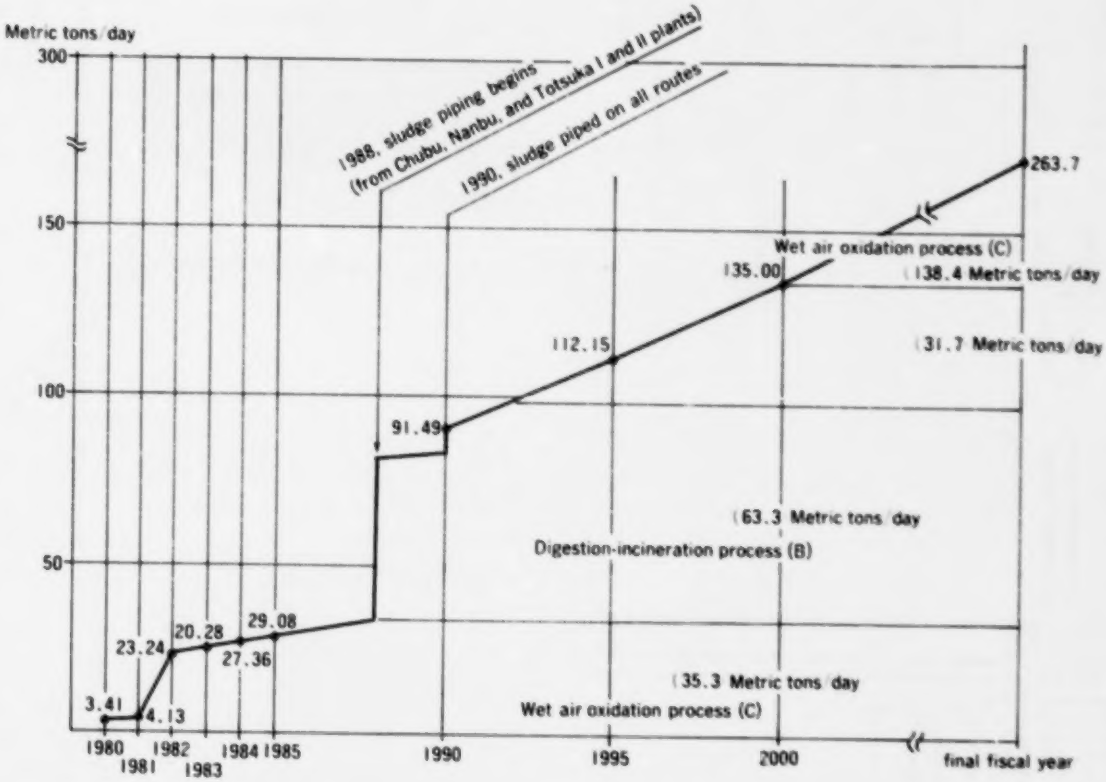


Figure 13. Nanbu Sludge Treatment Center construction plan

5.3 Energy use planning

One of the greatest advantages of centralized sludge treatment is that it allows energy to be effectively recovered. Both of Yokohama's sludge treatment centers were designed for optimum utilization of the energy derived from sludge.

The forms of energy recoverable from sludge--electric, mechanical, chemical and thermal--are shown in Figure 14. This figure also indicates rough rates of energy conversion. Analysis of the possible uses of energy recovered at the center revealed that thermal energy could be used to heat the digestion tanks and for local heating and air conditioning, and that mechanical energy could be used to drive the blowers and pumps at the center. The study showed these uses to be limited, however. Electricity, on the other hand, was found to offer a broader sphere of applications, including lighting and driving machinery. This suggested that the most effective system would be one that used digestion gas to generate electricity and made use of both this electric energy and the thermal energy derived from waste heat.

In light of this analysis, electric power generation using digestion gas was chosen as the basic method for both centers. Results of trial calculations concerning energy recovery with the digestion gas-fueled generator system in use at the Nanbu Sludge Treatment Center are described below.

The system centrifuges 95 metric tons of dry solids per day of incoming sludge to a concentration of 5 percent solids and subjects it to high-concentration digestion. Most of the resulting digestion gas is burned by a gas engine to generate electricity, and the waste heat from the gas engine is recaptured by an external heat exchanger and used to warm the digestion tank.

Table 9 shows the conditions under which the system's energy balance was calculated.

Figure 15 shows the heat balance in the digestion gas cycle. The data are grouped in three rows. The top row shows data for summer and the middle row for winter. The bottom row shows the average. On the average, 136×10^6 kcal/day are recovered in the form of digestion gas from the first-stage tank.

The heat required to warm the digestion tanks is the sum of the heat released by the tanks (10×10^6 kcal/day) and the heat required to heat the incoming sludge (29×10^6 kcal/day) or 39×10^6 kcal/day.

To recover enough waste heat from the gas engine-generator-heat exchanger system to warm the digestion tanks, it is therefore sufficient to supply the system with fuel equivalent in heat to about 89 percent of this, or 136×10^6 kcal/day. The remaining 11 percent of the digestion gas can be used as fuel in other systems or facilities, as auxiliary fuel for the vertical multistage incinerator or as fuel for the wet air oxidation system's deodorization equipment, for example. Because the multistage incinerator (capacity 100 metric tons/day) requires 23×10^6 kcal/day, the use of digestion gas reduces its fuel requirement by about 76 percent.

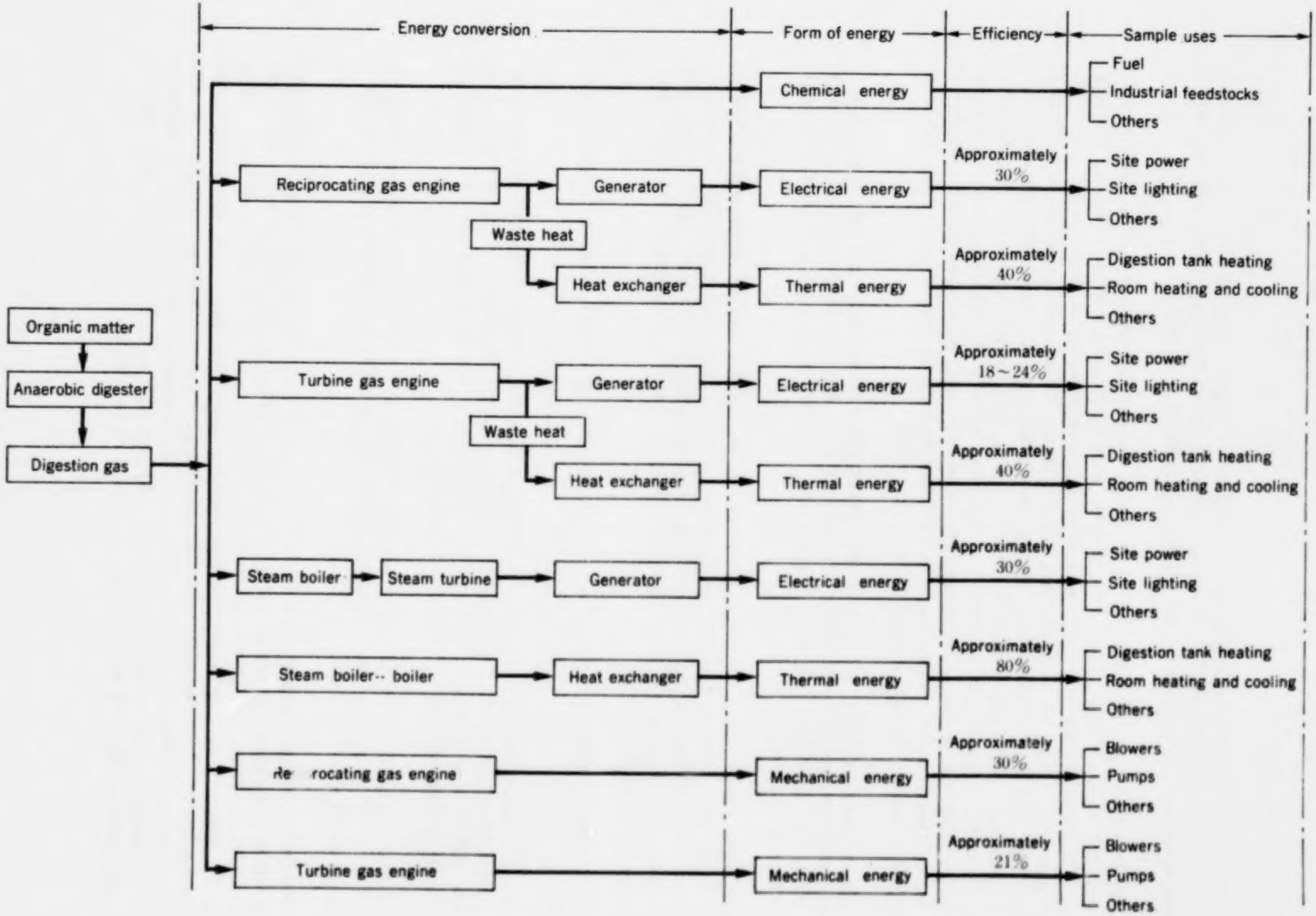

Energy conversion
Form of energy
Efficiency
Sample uses
Organic matter
Anaerobic digester
Digestion gas
Chemical energy
Fuel
Industrial feedstocks
Others
Reciprocating gas engine
Generator
Waste heat
Heat exchanger
Electrical energy
Approximately 30%
Site power
Site lighting
Others
Thermal energy
Approximately 40%
Digestion tank heating
Room heating and cooling
Others
Turbine gas engine
Generator
Waste heat
Heat exchanger
Electrical energy
Approximately 18 ~ 24%
Site power
Site lighting
Others
Thermal energy
Approximately 40%
Digestion tank heating
Room heating and cooling
Others
Steam boiler
Steam turbine
Generator
Electrical energy
Approximately 30%
Site power
Site lighting
Others
Steam boiler-- boiler
Heat exchanger
Thermal energy
Approximately 80%
Digestion tank heating
Room heating and cooling
Others
Re rocating gas engine
Mechanical energy
Approximately 30%
Blowers
Pumps
Others
Turbine gas engine
Mechanical energy
Approximately 21%
Blowers
Pumps
Others

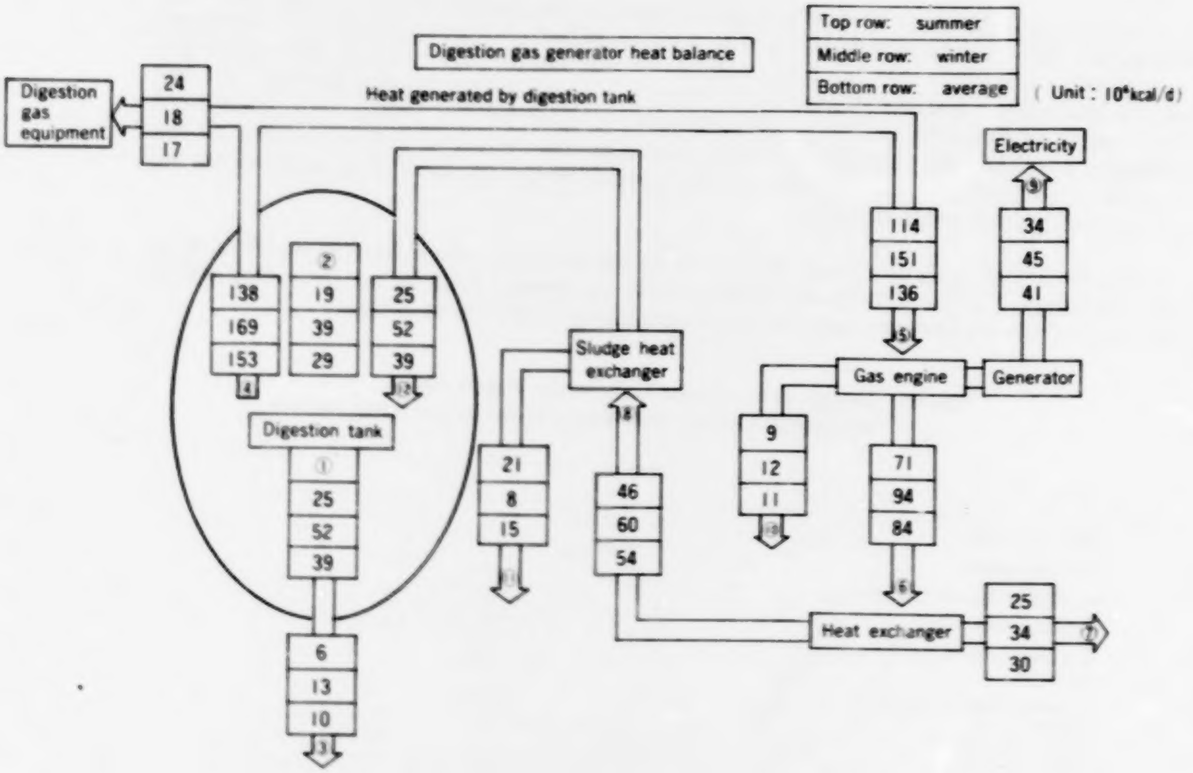
Figure 14. Uses for recovered energy

Turning now to heat recovery in the digestion gas-fueled generator system, 41 x 10^6 kcal/day (about 30 percent of the 136 x 10^6 kcal/day of heat supplied) is recovered as electricity, and 84 x 10^6 kcal/day (about 62 percent of the heat in the waste gas from the gas engine and the engine cooling water) is recovered as heat energy. Of the latter, about 46 percent, or 39 x 10^6 kcal/day, is used by the sludge and other heat exchangers to warm the digestion tanks. Although the boiler is not necessary to the heat energy balance, it was included in the system design to serve as a back-up during start-ups and system malfunctions.

The 41 x 10^6 kcal/day heat recovered in the form of electricity is equivalent to 47,600 kWh/day. This contributes substantially to energy savings by providing about 33 percent of the electricity used at the Nanbu Sludge Treatment Center.

Table 9. Energy balance conditions in the Nanbu Sludge Treatment Center's digestion gas-fueled generator system

Item	Value
Incoming sludge	95 DST/day
Incoming sludge	1,900m^3/day
Incoming sludge concentration	5%
Incoming sludge VTS	65%
Incoming sludge temperature (summer)	25℃
Incoming sludge temperature (winter)	12℃
Digestion period	30 days 1st stage : 20 days 2nd stage : 10 days
Digestion temperature	35℃
Digestion tank capacity	6,400m^3/tank x 9 tanks
Heating method	External heater
Volume of digestion gas generated	0.45m^3/VTS/kg of incoming sludge
Net calorific value of digestion gas	5,500 Kcal/Nm^3
Overall heat transfer coefficient	Air : K = 0.919Kcal/m^2/hr℃ Soil : K = 1.853kcal/m^2/hr℃
Ground temperature	16.2℃
Air temperature (summer)	26.2℃
Air temperature (winter)	4.9℃
Gas engine type	Electric ignition
Gas engine output	800 HP
Generator type	Three-phase AC
Generator output	500 KW x 4
Conversion efficiency of digestion gas engine-generator system	30%
Gas engine heat recovery rate (sludge warming heat)	40%


Digestion gas generator heat balance
Top row: summer
Middle row: winter
Bottom row: average
(Unit : 10^6kcal/d)
Digestion gas equipment
24
18
17
Heat generated by digestion tank
Electricity
⑨
114
151
136
34
45
41
138
169
153
②
19
39
29
25
52
39
④
⑫
Sludge heat exchanger
⑤
Gas engine
Generator
Digestion tank
①
25
52
39
21
8
15
⑧
46
60
54
9
12
11
71
94
84
⑩
⑪
⑥
25
34
30
Heat exchanger
⑦
6
13
10
③

Legend

Number	Item	Number	Item
①	Total digestion tank heat requirement	⑦	Heat lost by heat exchanger
②	Heat used to warm incoming sludge	⑧	Heat recovered from gas engine
③	Heat lost by digestion tank	⑨	Heat used to generate electricity
④	Heat generated by digestion tank	⑩	Heat lost by gas-fueled generator
⑤	Heat supplied to gas engine	⑪	Heat lost by sludge heat exchanger
⑥	Gas engine exhaust heat	⑫	Heat supplied to digestion tank

Figure 15. Nanbu Sludge Treatment Center digestion gas energy balance

5.4 Instrumentation

5.4.1 Basic structure of instrument systems

The sludge treatment centers are equipped with large-scale equipment--sludge receiving, thickening, digesting and digestion gas-fueled units--whose control is extremely complex.

For comprehensive integrated sludge control, the units must be organically interconnected. This is accomplished at the centers through a centralized supervision and distributed control system.

The data processing system at the heart of the supervision and control system consists of two side-by-side 32-bit computers for reliability and ease of maintenance. One of the computers is usually on standby.

Distributed local stations control the various complex automatic functions and perform the necessary computations. The local stations have microcontrollers and sequence controllers with data transmission capability to the main computer.

Fiber optic cables are used for data transmission between local stations and the main computer. They are linked by an N:M transmission method.

The system that pipes sludge from the sewage treatment plants to the center for centralized treatment also involves a remote data transmission link between the center and the various plants. The sludge piping system is an integral part of the centralized supervision and distributed control system. It is treated at the control level in the same way as the other local stations.

Figure 16 is a diagram representing the sludge treatment centers' instrumentation system.

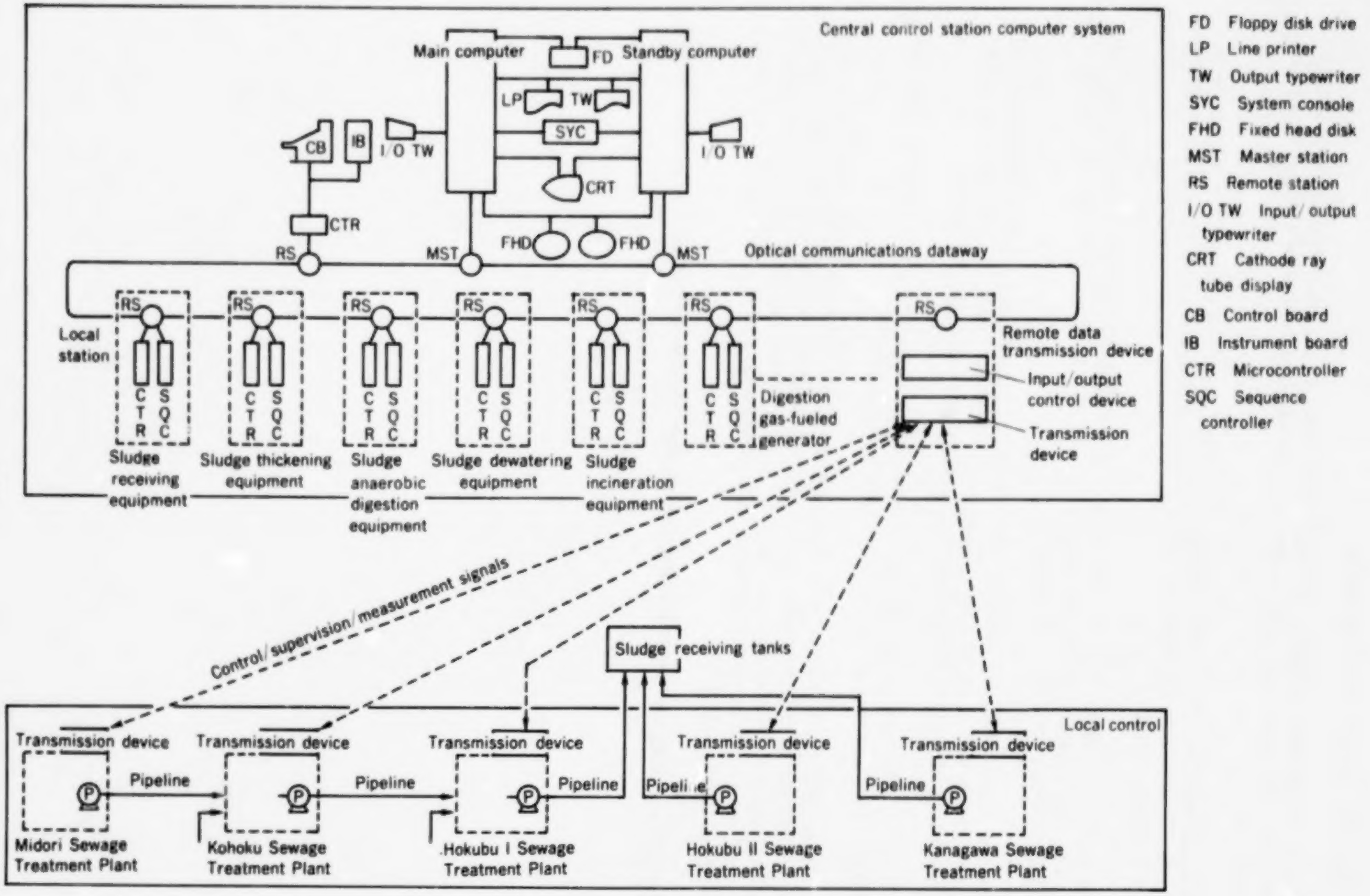

Central control station computer system
Main computer
FD
Standby computer
LP
TW
SYC
CRT
CB
IB
I/O TW
I/O TW
CTR
RS
MST
FHD
FHD
MST
Optical communications dataway
Local station
RS
C T R
S Q C
Sludge receiving equipment
Sludge thickening equipment
Sludge anaerobic digestion equipment
Sludge dewatering equipment
Sludge incineration equipment
Digestion gas-fueled generator
Remote data transmission device
Input/output control device
Transmission device
Control/supervision/measurement signals
Sludge receiving tanks
Local control
Transmission device
Pipeline
P
Midori Sewage Treatment Plant
Kohoku Sewage Treatment Plant
Hokubu I Sewage Treatment Plant
Hokubu II Sewage Treatment Plant
Kanagawa Sewage Treatment Plant
FD Floppy disk drive
LP Line printer
TW Output typewriter
SYC System console
FHD Fixed head disk
MST Master station
RS Remote station
I/O TW Input/output typewriter
CRT Cathode ray tube display
CB Control board
IB Instrument board
CTR Microcontroller
SQC Sequence controller

Figure 16. *Sludge treatment center instrumentation*

5.4.2 Sludge feed and intake system instrumentation

The sludge piping instrumentation at the completed Hokubu Sludge Treatment Center will now be described in greater detail.

a. Overview

Sludge produced at sewage treatment plants is pumped to the Hokubu Sludge Treatment Center through pipelines. To facilitate planned, efficient treatment of the sludge collected from these plants, the sludge pumps are remotely controlled from the sludge treatment center.

Thus the center also acts as the central control station for the system, with the various sewage treatment plants acting as local or site piping control stations.

The local facilities are equipped with microcontrollers and sequence controllers which receive operating instructions from the central control station and control the equipment according to fixed control procedures. The central and local control stations are linked by remote data transmission equipment using telephone lines.

Instructions concerning volumes of sludge to be sent and methods of transporting it are sent from the central control station to the local station via this remote data transmission equipment. The local microcontrollers take the instructions and convert them into specific control procedures for the sequence controllers which activate the pumps and valves.

Information concerning such things as the operating status of the pumps and valves at the local station, volumes of sludge being piped, amounts of solid matter and equipment problems is transmitted from the local station back to the central control station. It is received here in batches by input/output control equipment and relayed to the central computer system via the center's internal dataways.

At the center, the central computer system batch-treats data received from the local stations and displays them in diagrammatic, numeric and character form on the CRT displays which function as the man-machine interface.

b. Control

Sludge feed pumps can be controlled in three modes: monitoring sludge storage tank levels, presetting and timer setting. These modes determine the values at which control functions are activated from the central control station.

i. Level control

Tank levels can be controlled to achieve a balance between the sending and receiving side.

The sending pumps begin operation when storage tank levels on the receiving side fall below a predetermined value. Pumping continues until either there is no more sludge on the sending side or tank levels at the receiving side have reached a predetermined maximum level.

ii. Preset control

In this control method, total daily quantities and the overall frequency of sludge piping at a given sewage treatment plant are set in advance. Based on this setting, the microcontroller calculates quantities for a single sludge piping session and a piping schedule, then begins regular consecutive piping according to the program it has established.

Since the quantity of sludge piped in one day is determined according to its solid content, solid concentrations are measured and converted into equivalent sludge volumes. This preset control method is extremely effective in keeping track of the amounts of sludge piped daily.

iii. Timer control

The timer sets the frequency of sludge piping and the number of hours that the pipes are in operation for each piping session, then carries out piping consecutively. This method is a very effective in keeping track of piping time.

Once a piping session is over, secondary effluent is piped through the pipeline to flush it clean.

Piping operations can also be controlled from the central control station by manually starting and stopping the pumps.

c. Supervision

Information concerning the state of sludge piping equipment operation is sent by the remote data transmission instruments to the central control station.

At the central control station this information is input to the computer for observation on CRT displays and instrument boards. It is also recorded on output typewriters and recording instruments.

Table 10 shows the main items observed.

d. Measurement

Data readings at the sending facilities on piped sludge volumes, sludge concentrations, sludge storage tank levels and so forth are sent by the remote data transmission instruments to the central control station.

At the central control station, this information is input to the computer and displayed on the CRT display, as well as continuously recorded by recording instruments.

e. Detection of pipe clogging and leakage

Obstructions in the sludge pipes are detected by logic circuits as shown in Figure 17. These conclude a pipe is blocked if a pump's outlet pressure exceeds a certain value and sludge flow either reaches a certain minimum value or drops suddenly. When an obstruction is detected, the feed pump is immediately shut off and an obstruction warning buzzer sounded.

Table 10. Supervision items

Equipment	Item
1. Feed pump	1. Automatic/manual 2. Preparations complete 3. Start/stop/trouble
2. Outlet valve	Closed/open/trouble
3. Draw valve	Closed/open/trouble
4. Sludge storage tank	High/low
5. Control mode	1. Level control 2. Timer control 3. Preset control
6. Electric failure	Ground fault, overvoltage, undervoltage, temperature rise, control power failure, sequence controller trouble
7. Other	1. Fire 2. Pipeline abnormality 3. Transmission abnormality 4. Building or ancillary facility trouble

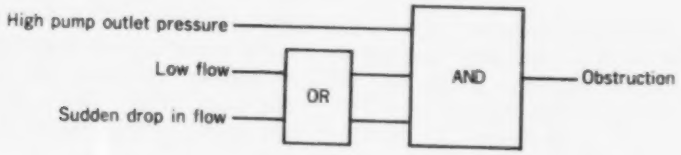

High pump outlet pressure
Low flow
Sudden drop in flow
OR
AND
Obstruction

Figure 17. Obstruction logic circuit

Sludge pipe leakage is also detected by logic circuits as shown in Figure 18. They conclude a pipe is leaking if its outlet pressure falls below a certain level and sludge flow either exceeds a certain normal value or registers a sudden increase. On detection of a leak, the feed pump is immediately shut off and a leakage warning buzzer sounded.

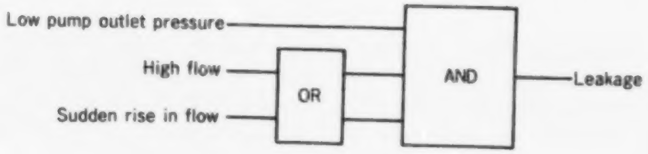

Low pump outlet pressure
High flow
Sudden rise in flow
OR
AND
Leakage

Figure 18. Leakage logic circuit

6. Conclusion

In the past, sludge treatment in Yokohama was performed at the sewage treatment plants where the sludge was produced. A plan is being implemented in stages, however, to centralize sludge treatment at two sludge treatment centers located on the coast. Centralized treatment is considered necessary in view of the growing quantities of sewage and sludge requiring treatment, the impact of treatment on the environment surrounding sewage treatment plants, and the need to increase the efficiency of sludge treatment and disposal.

There are, however, disadvantages as well as advantages to centralized sludge treatment. Problems requiring solution include the following:

a. Transport of sludge over long distances

Linking the eleven sewage treatment plants requires 88 kilometers of pipe. The magnitude of this network makes detection of leaks and other problems extremely difficult.

b. Stability and safety of the sludge pipe network

The network is currently designed for single-lane pipelines. This leaves much to be desired in terms of system stability and safety in the event of earthquakes or other disruptions. One possible remedy is to lay double piping between all the sewage treatment plants and to develop the system into a network of two-way links between plants. Ways must also be found to use all the facilities at all of the plants in the event of earthquakes or other emergencies.

c. Recycling of process water from sludge treatment centers

Recycling of process water was not a problem in the past when sludge was treated at individual sewage treatment plants. However, at the sludge treatment centers, the enormity of the recycling load is expected to cause problems. In response, experiments have been performed in search of methods to treat and recycle process water, including pressurized aeration, deep shaft aeration, and other biological processes. The results showed that these methods yielded an unsatisfactory 50 percent reduction in chemical oxygen demand (COD_{Mn}).

These are the problems we face. Several years will be required before the sludge treatment centers are completed, and we hope that in the meantime new sludge and recycled process water treatment technologies will be developed and put to use. We wish to be able to adapt flexibly to such new technologies and thereby establish a more efficient sewage and sludge treatment system.

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Tenth United States/Japan Conference
on Sewage Treatment Technology

ODOR CONTROL IN MUNICIPAL WASTEWATER TREATMENT PLANT

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1. PRESENT STATUS OF ODOR PROBLEMS IN JAPAN

It dates back to the 1950s when offensive odors started to be a social problem in Japan. In the 1960s when Japan achieved a miraculous economic growth, odor problems took on a pandemic phase. In 1966, statistical survey was initiated about the odor complaint cases; the number of cases filed with the public bodies in 1966 was 3,500.

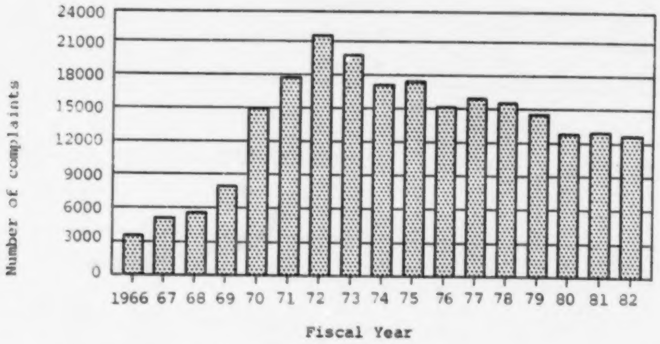


Fig. 1 Annual changes of complaints about offensive odors

Fig. 1 shows the transition of odor complaint cases from 1966 to 1982. As demonstrated, the number of cases rose six times in the first six years from 1966 till 1972. Concerned about the situation, the Government put into effect the Offensive Odor Control Law in 1972 to authorize the local municipalities to control odor concentrations. From 1973 afterward, the number of complaint cases had been on the decline; in 1982, it fell to about 13,000.

At present, local municipalities are empowered by the offensive Odor Control Law to control eight odorous substances at the metes and bounds of every odor source, as shown in Table 1.

According to the Law, the prefectural governor is authorized to determine the regulated areas and standard odor concentrations after hearing out the opinion of local municipalities. As of 1982, 1,256 municipalities or 39% of all, have regulations by the Offensive Odor Control Law.

Ever since the enforcement of the Law, the complaint cases have been decreasing. According to the statistics for FY 1982, livestock farming and service industry led the list of odor complaints with 28.1% and 19.8%, respectively as seen in Fig. 2.

Table 1 Odorous substance limits according to Offensive Odor Control Law

Odorous substances	Limits (air-borne concentration at plant boundary (ppm))
Ammonia	1 to 5
Methyl mercaptan	0.002-0.1
Hydrogen sulfide	0.02-0.2
Dimethyl sulfide	0.01-0.2
Trimethylamine	0.005-0.07
Dimethyl disulfide	0.009-0.1
Acetoaldehyde	0.05-0.5
Styrene	0.03-20

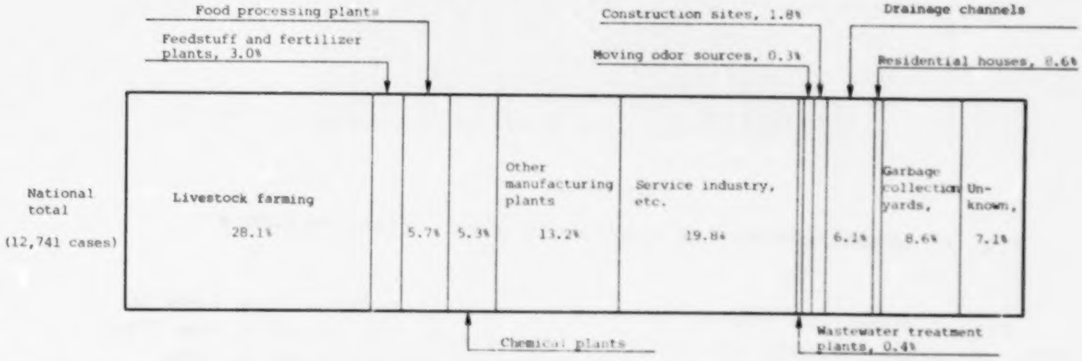


Fig. 2 Percentage distribution of sources blamed for offensive odors

The number of complaint cases against the wastewater treatment plants was 49, or 0.4%.

While the siting conditions are important for the prevention of odor problems, there are many wastewater treatment plants adjacent to residential quarters that have obviated odor problems with proper measures. One of the most viable measures is to cover up the odor source. In Japan, 70% of sludge thickners and 75% of grit chambers are installed indoors or covered. Thirty percent of rather larger facilities such as aeration tanks and primary settling tanks are also covered with covers made of FRP and other plastic materials. At present, the open yard drying beds have been replaced almost totally by mechanical sludge dewatering processes installed indoors. Some 13% of dewatering machines are covered. Sixty-nine percent of sludge cake hoppers are totally sealed.

The wastewater treatment facilities equipped with deodorizing equipment are 38% for grit chambers, 43% for sludge thickners, and 58% for sludge dewatering machines.

About a decade back, it was a common practice to scrub odors simply with a sodium hydroxide solution or water. Today, many wastewater treatment plants use absorbents such as activated carbon and ion exchange resin or sodium hypochlolide to remove odors.

Soil absorbability and biological decomposing activities are used to remove odors, as well as a process in which foul air is blown into activated sludge reactor for deodorization.

With the progress of deodorization technology, there are few odor problems with the existing wastewater treatment plants, except for some special cases.

2. PURPOSES AND OBJECTIVES OF RESEARCH AND STUDY

In Japan, the deodorization measures have been working well. Many of technologies applied in deodorization are originated from paper and pulp industry and night soil treatment plants. In the municipal wastewater treatment plants, malodorous substances are lean in concentration as compared with those in paper and pulp mills or night soil treatment plants, but the volume of air to be handled is quite large.

The study was prompted to review deodorization measures for the wastewater treatment plants according to the following topics.

1) To determine characteristics and volume of odor substances from each of the processes, including grit chamber, aeration tank, sludge thickner, and sludge incinerator, and to make clear their organoleptic relationships quantitatively.

2) To study the method of predicting the volume of odorous substances and predicting the dispersion into the atmospheric environment.

3) To identify the covering and ventilation methods suitable for each of the processes.

4) To study the optimum deodorizing methods.

3. ODOROUS SUBSTANCES AND THEIR SOURCES IN THE PLANT

3.1 Odor Measurement

The data of odor measurement here were obtained by following methods.

(1) Odor concentration

The concentration of an odor was measured on an following organoleptic test. First, two scent bags are purged with the room air cleaned through activated charcoal, and another bag is prepared purged with an odor which is diluted with clean air. These bags are then numbered 1 through 3 at random, and are sniffed at by the members of a test panel, all having a sound sense of smell, through a nose cone. They are asked to take down the number of the bag that stank most. If the panel answered correct, the dilution ratio is increased a notch.

The test closes when the dilution ratio has begun to confound the panelists.

(2) Odor strength

This is also measured by an organoleptic method, and the odor strength is expressed in six grades as shown in Table 2.

Table 2 Six-grade of odor strength

Odor strength	Description
0	No odor
1	Barely detectable (detection threshold level)
2	Barely identifiable level of odor (identification threshold level)
3	Easily identifiable
4	Strong odor
5	Very strong odor

(3) Ammonia

Several milliliters to several hundred liters of ammonia gas is blown into 20 milliliters of 0.5% aqueous solution of boric acid as an absorbent at a rate of 1 to 2 liters a minute, the volume of ammonia gas to be supplied being dependent on the concentration of ammonia gas. The odor strength is then measured according to the indophenol method.

(4) Sulfur compounds (hydrogen sulfide, methyl mercaptan, methyl sulfide, methyl disulfide)

The odor collected in a gas collecting bag is passed through a sample concentrating tube which is refrigerated with liquefied nitrogen. In this way, sulfur compounds are concentrated and subjected to gas chromatography for fractionation.

(5) Lower fatty acids

An odor is passed through a column filled up with glass beads coated with strontium hydroxide for trapping lower fatty acids by reaction.

Formic acid is then added to the column, and the eluate containing lower fatty acids is fractionated through a gas chromatograph equipped with a flame ionization detector.

(6) Aldehydes

Aldehydes are trapped in a solution of 2,4-dinitro-diphenyl hydrazine, and are extracted with carbon tertachloride. The extract is then concentrated 20 times and gas-chromatographed.

3.2 Odorous Substances

A wide spectrum of malodorous substances which have been identified by the survey since 1976 to be present at a level higher than the olfactory threshold value are as listed in Table 3.

Table 3 Substances associated with odors from wastewater

Classification	Chemical formula	Name	Threshold value, ppm	Odors like:
	H_2S	hydrogen sulfide	0.00041	Addled eggs
	NH_3	Ammonia	1.54	Pungent smell
Mercaptans (C_nH_{2n+1}-SH)	CH_3SH	methylmercaptan	0.00007	Pickled raddish
Sulfides (R-S-R')	$(CH_3)_2S$	dimethyl sulfide	0.003	Putrid cabbage
	$(CH_3)_2S_1$	dimethyl disulfide	0.0022	Laver
Aldehydes (R-CHO)	HCHC*	formaldehyde	0.41	Pungent smell
	CH_3CHO	Acetoaldehyde	0.0015	Irritating smell of ether
	C_2H_5CHO	Propionaldehyde	0.00067	Heavy smell of formaldehyde
	$(CH_3)_2CHCHO$	iso buthylaldehyde	0.00055	
	C_3H_7CHO	n-buthyladehyde	0.00029	
	$(CH_3)_2CHCH_2CHO$	iso-barelaldehyde	0.000069	
	C_4H_9CHO	n-bareladehyde	0.00041	
Ketons (R-CO-R')	CH_3COCH_3*	Aceton	30	Smell of solvent
Indoles	C_9H_9N	Skatole	0.0000056	Night soil
	C_8H_7N	Indole	0.0003	Mixture of chloric acid and acetic acid
Fatty Acids (R-COOH) Cl - C	CH_3COOH	Acetic Acid	0.0042	Vinegar
	C_2H_5COOH	propionic acid	0.0057	Nasty smell, sweetened smell of acetic acid
	C_3H_7COOH	Butyric acid	0.00019	Smell of sweat (cheese)
	$(CH_3)_2CHCOOH$	iso butyric acid	0.0015	
	$(CH_3)_2CHCH_2CHO$	iso veleric acid	0.000078	
	C_4H_9COOH	n-veleric acid	0.000037	
Oxidised Nitrogen	NO_2	Nitrogen dioxide	0.03	
Amines	$(CH_3)_3N$	Trimethylamine	0.000027	Stale fish

Note: The concentrations of the chemicals marked with an asterisk correspond to strength 1 according to the six-grade odor classification method. The chemicals with no asterisk are according to the three-scent bag comparison method.

Observed odor concentrations and the estimated odor concentrations calculated by dividing the concentrations of detected substances by respective threshold values are correlated for each wastewater treatment facility as shown in Figs. 3 through 9.

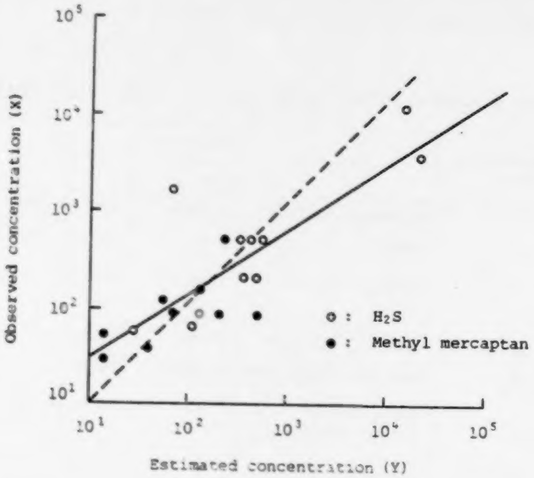


Fig. 3 Observed vs. estimated odor concentrations (grit chamber)

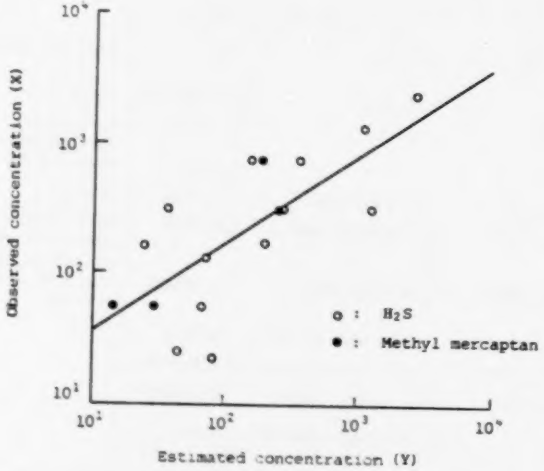


Fig. 4 Observed vs. estimated odor concentrations (primary setting tank)

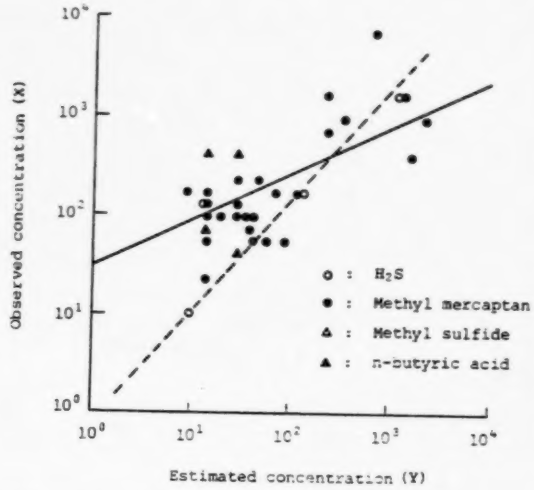


Fig. 5 Observed vs. estimated odor concentrations (aeration tank)

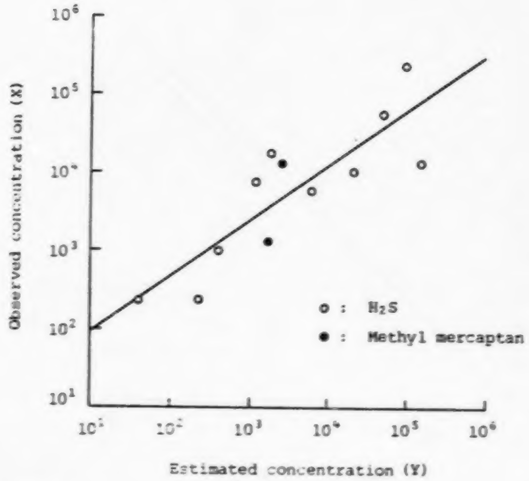


Fig. 6 Observed vs. estimated odor concentrations (sludge thickner)

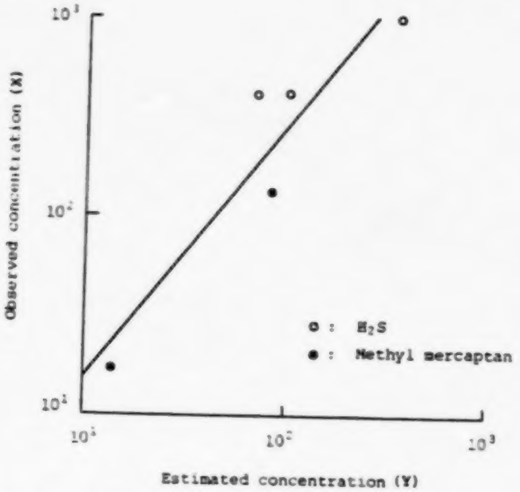


Fig. 7 Observed vs. estimated odor concentrations (sludge elutriation tank)

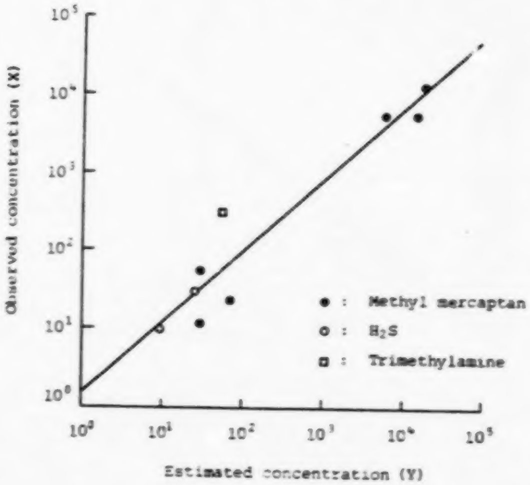


Fig. 8 Observed vs. estimated odor concentrations (dewatering equipment)

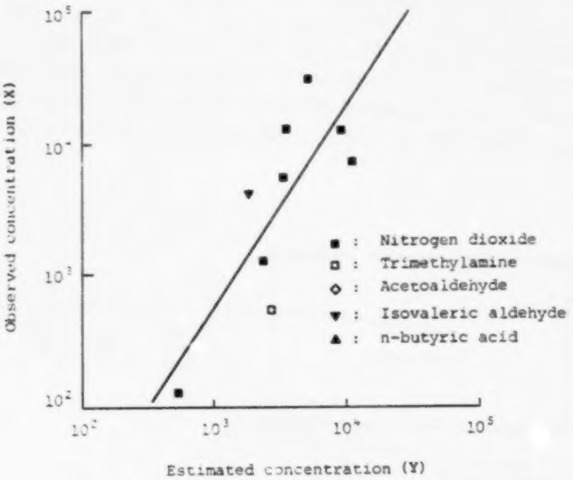


Fig. 9 Observed vs. estimated odor concentrations (incinerator)

As grit chamber, primary settling tank, sludge elutriation tank and sludge thickner are in an anaerobic condition, hydrogen sulfide and methyl mercaptan are major odor substances common to them. The above two substances are also ascribable to the odor from the sludge dewatering machines. When digested sludge is dewatered with lime, alkaline odors like trimethyl amine are produced which, together with the irritating odor of ammonia, produce a peculiar odor.

In the aeration tank, methyl mercaptan is most blamable, and its observed concentration often exceeds the estimated one; this suggests that there may be some odor-contributing substances other than chemically analyzed.

The sludge incinerator is found to give off nitrogen dioxide and aldehydes, and requires odor-preventive measures different in nature from those for other facilities.

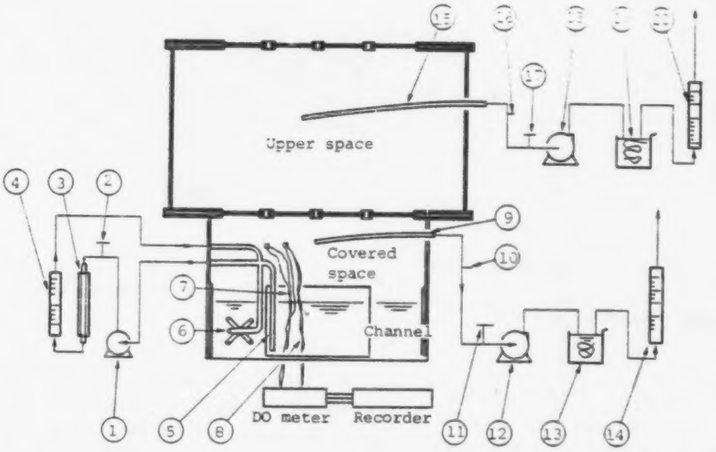
4. MECHANISM OF ODOR DEVELOPMENT

The propagation of smell from the sewage or sludge into air can be explained as a phenomenon of mass transfer between gas and liquid. The open channels of grit chambers, weirs of primary settling tank, aerated channels of aeration tank, etc. offer chances of transfer. Usually, odors ar cut off at source by cover-up, and the concentrations of malodorous substances in the open air are governed by the rate at which the freeboard space of the covered odor source is ventilated.

4.1 Study on Ventilation Rate

For a covered aeration tank, for example, experience shows that the ventilation is satisfactory if it is 20% to 30% greater than the aeration rate.

The leakage from the primary settling tank, sludge thickner, etc., can be prevented by keeping their freeboard space in a vacuum. When there is a temperature difference between the freeboard and the open air, the thermal convection effect asserts itself, and the ventilation rate required to prevent odor release should naturally be controlled. Fig. 10 shows a system used for a series of ventilation tests.



① Water circulation pump
② Water flow regulating value
③ Cooling pipe
④ Flowmeter
⑤ Water suction port
⑥ Water outlet
⑦ DO sensor
⑧ Temperature compensating electrode
⑨ vacuum pipe (covered space)
⑩ Odor sampling cock (covered space)
⑪ Flow control valve (covered space)
⑫ Fan (covered space)
⑬ Cooling unit (covered space)
⑭ Flowmeter (covered space)
⑮ Vacuum pipe (upper space)
⑯ Sampling cock (upper space)
⑰ Flow control valve (upper space)
⑱ Fan (upper space)
⑲ Cooling unit (upper space)
⑳ Flowmeter (upper space)

Fig. 10 Ventilation test facility

The water temperature (Tw) and the upper space temperature (Ts) were measured during the test. The air velocity below which methyl mercaptan would not be carried into the upper space was taken as the critical air velocity (u).

Fig. 11 shows the relationship between the critical air velocity and the temperature difference (Ts - Tw).

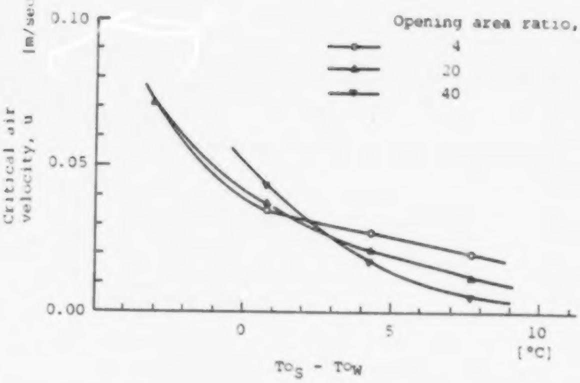


Fig. 11 Relationship between critical air velocity and temperature difference

Indicating that, when Tw gets lower than Ts, the critical air velocity to prevent leakage get higher. As in the sludge thickner, dewatering machine, and other machines can be covered by the opening area ratio of less than 1%, ventilation rate can be held practically small enough at the value as 0.3 m/sec. At the grit chamber and primary settling tank, the inlet and outlet are exposed, and the ventilation rate needs to be considerably large. However, this ventilation rate can be reduced successfully by covering such exposed parts with a canvas sheet to minimize the opening.

Using the test system illustrated in Fig. 10, we studied the overall mass transfer coefficient of oxygen and carbon disulfide from the water into or to the air.

Fig. 12 shows the relationship between the ventilation frequency and the overall mass transfer coefficient (K_{LY}) obtained by the test at the range of ventilation frequency between 4 and 57 times/hr. While the K_{LY} value of carbon disulfide remained constant, that of oxygen tended to go down a little with increase in the ventilation frequency, it looked almost constant. It is estimated from above result that the overall mass transfer coefficient of odor substance assumes a characteristic value of the channel under normal conditions.

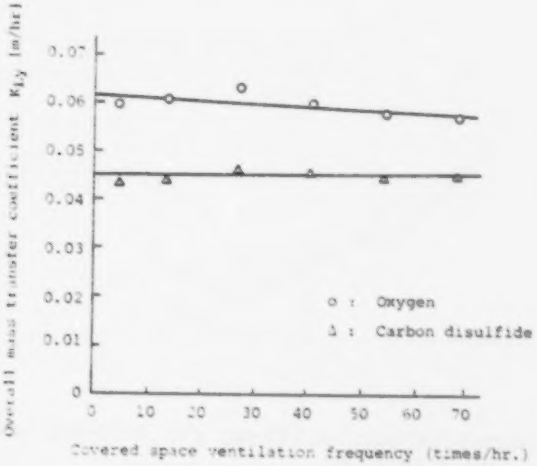


Fig. 12 Relationship between ventilation frequency and overall mass transfer coefficient

4.2 Overall Mass Transfer Coefficient at the Channel and Weir

Kyosai1) proposed a method of estimating the overall mass transfer coefficients of odorous substances from the mass transfer coefficient of oxygen for which volumes of field data at channels, weirs and aerated channels are available. The overall mass transfer coefficient of a volatile substance not dissolved in the water is expressed by Eq. (1) as a function of the overall mass transfer coefficient of oxygen under the following conditions.

i) The transfer coefficient of the volatile substance at the gas layer film is negligible as compared with that at the liquid layer film.

ii) The volatile substance liberated into the air is dissipated immediately, and its partial pressure in the air is negligibly small.

iii) The liquor is a complete mix situation.

$$K_{LY}/K_{LZ} = (V_{CZ}/V_{CY})^{0.3144} = \beta_{Y,Z} \quad \text{........} \quad (1)$$

where K_{LY}: overall mass transfer coefficient, Y (m/hr.); K_{LZ}: overall mass transfer coefficient of reference substance, Z (oxygen here) (m/hr.); V_{CZ}: critical volume of substance, Z (m^3/h-mol); V_{CY}: critical volume of substance, Y (m^3/g-mol); $\beta_{Y,Z}$: a constant relating to Y and Z.

The system shown in Fig. 10 was used as a channel model.

Tap water laced with 0.5 mg-Co/ℓ in the form of cobalt chloride was added with sodium sulfite to reduce the dissolved oxygen concentration to about 2 mg/ℓ. This water was then added with, carbon disulfide, hydrogen sulfide, methyl mercaptan, methyl sulfide and methyl disulfide, at a concentration of 10 mg/ℓ to obtain a test sample.

The test sample was circulated at a rate of mainly 11 cm/sec. or 23 cm/sec. while the ventilation was made at a rate of 4 times an hour. Dissolved oxygen was measured with a DO meter, and the decline of the test subject was measured by a gas-chromatograph by real-time sampling. The substances other than carbon disulfide and methyl sulfide inhibited the oxidation of sodium sulfite, and the mass transfer coefficient of oxygen measured with such oxidation-inhibiting substances was discarded, and the transfer coefficient obtained in the measurement of carbon disulfide alone was accepted.

Table 4 shows the mass transfer coefficients of various substances measured. The value of β_{Y,O_2} obtained from K_{LY} and K_{LO_2} was 82 to 116% of the value calculated based on the critical volume proving that Kyosai's method is good for practical use.

Table 4 $\beta_{Y,Z}$ determined from the measurement of mass transfer coefficients of sulfur compounds and oxygen, and theoretically determined $\beta_{Y,Z}$

Substance	Circulating velocity	Critical volume	Mass transfer coefficient		Ratio of mass transfer coefficients, $\beta_{Y,Z}$		Measured/ calculated
	cm/sec.	cm/mol	K_{LY} (m/hr.)	$K_{L,20}$ (m/hr.)	Measured	Calculated	%
Carbon disulfide	3.3 - 23	170	0.0390*	0.0519*	0.0751	0.76	98
Hydrogen sulfide	23	98.5	0.0666	0.0630**	1.06	0.912	116
Methyl mercaptan	11 23	145	0.0333 0.0453	0.0377** 0.0630*	0.883 0.719	0.807	109 89
Methyl sulfide	11 23	201	0.0272 0.0483	0.0377** 0.0630	0.721 0.767	0.728	99 105
Methyl disulfide	11 23	260	0.0209 0.0307	0.0355 0.0557	0.589 0.551	0.672	88 82
Oxygen	-	73.4	-	-	-	-	-

* Average value of 17 cases with different velocities
** Measured value for carbon disulfide

The same test was conducted on a weir model. The model used is outlined in Fig. 13.

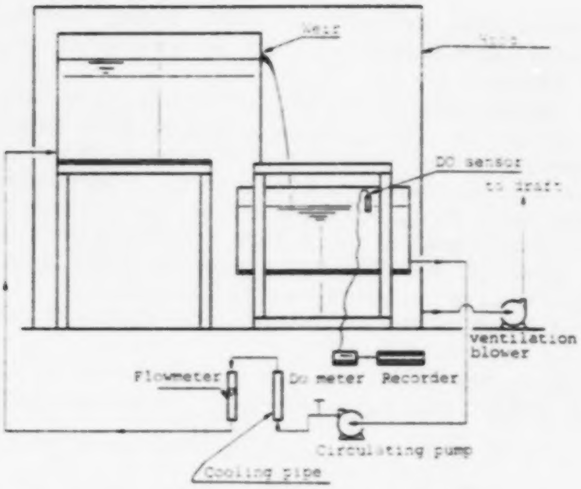

Weir
Hood
DO sensor
to draft
ventilation blower
Flowmeter
DO meter
Recorder
Circulating pump
Cooling pipe

Fig. 13 A schematic view of mass transfer coefficient measuring system with weir

The weir was rectangular with a crest width of 150 mm and the head on the crest was set in the range of 75 to 350 mm. The overflow loading was changed over a range of 58 to 285 m^3/m.day. The depth of the waterfall was fixed at 50 mm.

This system was covered with an acrylic resin covering, and ventilated 7.6 times an hour. According to the same procedure as with the channel, carbon disulfide test was conducted. β_{Y,O_2} was determined as the ratio of the mass transfer coefficient per weir of oxygen, K_{2,O_2} (1/day) to that of test substance, $K_{2,Y}$ (1/day). The relationship between the mass transfer coefficient of oxygen and β_{Y,O_2} is given in Fig. 14.

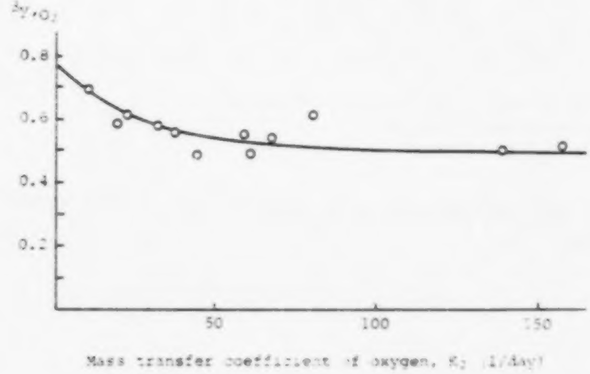


Fig. 14 Mass transfer coefficient of oxygen, and β_{Y,O_2} (carbon disulfide)

It was found that the larger the mass transfer coefficient of oxygen (K_{2,O_2}), the smaller the value of β; and that the value of β levels off when K_{2,O_2} is 50 (1/day) or greater. At the weir, mass transfer is considered to take place at the gas-liquid contact layers as offered by bubbles at the waterfall basin. The change of β may be explained by the fact that the hypothesis ii) above does not hold good for the bubbles.

Hydrogen sulfide and other substances were also measured with K_{2,O_2} set at 37.3 and 60.8 (1/day), to determine β_{Y_2,O_2}. Table 5 shows the calculated values of β and the measured values of β. Observed values are close to the calculated values. On the other hand, the observed values are 30 to 70% of the calculated values. We compared a grit chamber as a typical open channel with a primary settling tank offering a typical weir with respect to the mass transfer rate of hydrogen sulfide, and found that the mass transfer rate was far and away greater at the weir than the channel, with the ratio of 1 to 330.

Table 5 Comparison of channel and weir in terms of B value

Substance	Calculated value $\beta_{Y,O_2} = \frac{V_{C,O_2}}{V_{CY}}$	Channel test		Weir test	
		Measured value $\beta = K_{LY}/K_{L,O_2}$	Measured/ calculated	Measured value $\beta' = K_{2Y}/K_2$	Measured/ calculated
Carbon disulfide	0.764	0.750	0.982	0.563 0.493	0.737 0.645
Hydrogen sulfide	0.912	1.06	1.16	0.375	0.411
Methyl mercaptan	0.807	0.883 0.719	1.09 0.891	0.439 0.465	0.544 0.576
Methyl sulfide	0.728	0.721 0.767	0.990 1.05	0.397 0.432	0.545 0.593
Methyl disulfide	0.672	0.589 0.551	0.876 0.820	0.229 0.207	0.341 0.308

Note: Two values appear in a box. For the channel test, the upper and lower values correspond to 0.0377 m/hr. and 0.063 m/hr. of K_{LO_2}, respectively. For the weir test, the upper and lower values correspond to 37.3 (ℓ/day) and 60.8 (ℓ/day) of K_{2,O_2}, respectively.

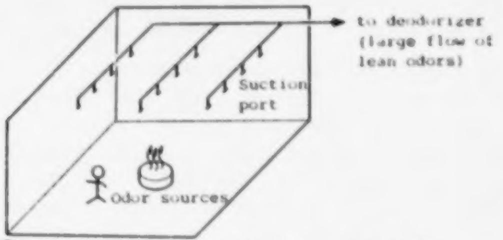
Even in the open channel, bubbles offer a major liquid-gas interface, and are considered to cause the same condition as with the weir.

5. DEODORIZATION TECHNOLOGIES

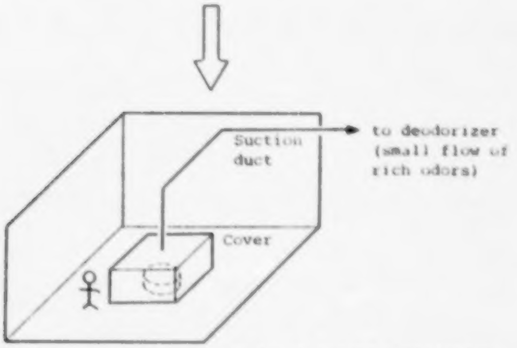
5.1 Odor Collection

An example of odor collection method is illustrated in Fig. 15. To collect and deodorize the room air as in Fig. 15-①, the volume of air to be collected becomes bulky, but the concentration of odors in the exhaust becomes lean.

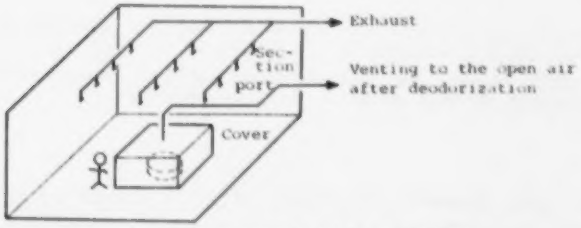
A solution to this problem is naturally to cover the odor source for facilitating local forced ventilation. If accommodation quarters such as administration office are present in the same building that has an odor source, it may be necessary to ventilate the accommodation space as well as the covered odor source as illustrated in Fig. 15-③.


to deodorizer
(large flow of
lean odors)
Suction
port
Odor sources

① Collection of odors from a spacious room


to deodorizer
(small flow of
rich odors)
Suction
duct
Cover

② Collection of odors by local forced ventilation


Exhaust
Sec-
tion
port
Venting to the open air
after deodorization
Cover

③ Where accommodation space and odor source are present in the same building

Fig. 15 Odor collecting methods

Several forms of covers for odor sources are shown in Fig. 16. A simple cover is suitable for outdoor plants such as sludge thickners. Where odors are to be collected from sources in a building, a double cover (double decked cover) is preferrable.

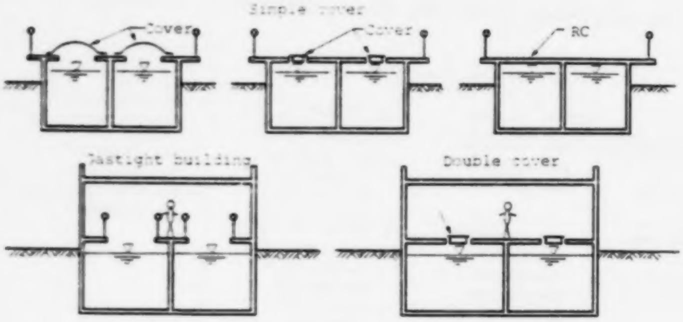


Fig. 16 Covering methods

The lifting equipment for grit chamber, screens, dewatering machines, etc. may defy covering depending on their types as the workability may be impaired by the covering. Thus, the types of covering and the feasibility of covering should be studied with care.

As the test results in Chapter 4 suggest, the forced ventilation for the covered freeboard will be enough if the air velocity at the opening is about 0.3 m/sec. For those facilities like sludge thickner, it will be advisable to install a draft tube of about 50 mm in diameter in order to check excessive depressurization within the cover.

5.2 Deodorization

There are four major deodorization methods available for the wastewater treatment plants.

They are: a wet chemical method using an aqueous solution of sodium hydroxide or sodium hypochlorite; an absorption method using activated carbon or ion exchange resin; an incineration to burn out odors; and a biological method using soil microorganisms or activated sludge. Table 6 is a classification of the methods recommended for applications. While the wet chemical method can be applied at a large ventilation rate, chemicals to be used must be selected properly.

Table 6 Deodorizing systems proposed for wastewater treatment facilities

	Odors from wastewater and sludge		Odors from sludge drying and incineration processes
	100 m^3/min.up	up to 100 m^3/min.	
High-concentration odors (concentration: 10,000 up)	o Wet chemical scrubbing + Adsorption	o Direct burning o Wet chemical scrubbing + Adsorption	o Direct burning o Wet chemical scrubbing o Contact reduction by ammonia
Medium-concentration odors (concentration: 1,000 to 10,000)	o Wet chemical scrubbing + Adsorption o Adsorption (combined use of chemical and physical adsorbents) o Odor removal by soil o Oxidation by aeration	o Direct burning o Wet chemical scrubbing + Adsorption o Adsorption (combined use of chemical and physical adsorbents) o Odor removal by soil o Oxidation by aeration	
Low-concentration odors (concentration: up to 1,000)	o Wet chemical scrubbing o Wet chemical scrubbing + Adsorption o Absorption o Odor removal by soil o Oxidation by aeration		o Wet chemical scrubbing o Contact reduction by ammonia

Table 7 summarizes empirical data obtained with respect to scrubbing efficiency. There are a wide variety of absorbents for the adsorption method. In the wastewater treatment plant, however, activated carbon and chemical absorbents lead the others. Activated carbon is available in two types.

Table 7 Recommendations on scrubbing methods to follow for successful deodorization of various odor substances (empirical data)

Chemicals / Compounds	Water scrubbing	Acid scrubbing	Alkali scrubbing	Alkaline sodium hypochlorite scrubbing	Neutral sodium hypochlorite scrubbing	Wet reduction scrubbing
Ammonia	o	◎	x	o	x	x
Trimethylamine	Δ	◎	x	Δ	x	x
Hydrogen sulfide	Δ	x	o	◎	o	x
Methyl mercaptan	x	x	x	◎	◎	x
Methyl sulfide	x	x	x	◎	◎	x
Methyl disulfide	x	x	x	Δ	◎	x
Acetoaldehyde	x	x	x	x	x	o
Styrene	x	x	x	x	x	x
Clhoride	x	x	◎	x	x	◎
Oxygen dichloride	Δ	x	◎	x	x	◎

* Conditions: Space velocity: 1 m/sec.; contact time: 1 sec.; gas/liquid ratio: 2 ℓ/m^3

◎: Removal efficiency, 90 to 100%
o: Removal efficiency, 70 to 90%
Δ: Removal efficiency, 30 to 37%
x: Removal efficiency, 0 to 30%

One for physical adsorption, the other coated by acid, alkali or oxidizing agent to meet the type of odor substance to be removed. Table 8 shows the adsorption capacity of the various types of activated carbon.

Table 8 Adsorption capacity of activated charcoal

Activated charcoal / Malodorous substance	Plain charcoal for physical adsorption	Special activated charcoal for chemically enhanced selective adsorption		
	Neutral	A (acid)	B (alkaline)	C (oxidative)
Ammonia		◎		
Trimethylamine		◎		o
Hydrogen sulfide	o		◎	o
Methyl mercaptan	o		◎	◎
Methyl sulfide	o		o	◎
Methyl disulfide	o		o	◎
Acetoaldehyde	◎			
Styrene	◎			

◎: Excellent
o: Good

Three different types of activated carbon - one for acid odors (hydrogen sulfide and methyl mercaptan), one for alkali odors (ammonia, trimethylamine), and one for neutral odors (methyl sulfide, methyl disulfide) - are used in multiple layers.

Incineration is effective in burning out high-concentration odors of small ventilation rate, and is not so common in the wastewater treatment plant. The biological method using soil microorganisms has come to stay mainly in the livestock rearing industry for deodorizing animal wastes. In recent years, it has become increasingly appealing for wastewater treatment plants, though it has some problems that a wide tract of land is required and that preliminary tests are necessary because the optimum gas feed rate varies depending on the type of soil. If these problems were solved, the method will spread for its simple operation and maintenance and high deodorizing efficiency. Table 9 shows the design particulars and operating conditions of a subterraneous deodorizing system at the Tarumi Wastewater Treatment Plant, Kobe. The land area required for the processing of odors at a rate of 1 Nm/min. is 3.7 m^2.

Fig. 17 shows the structure of this system. The deodorizing performance of Tarumi's system is quite impressing as demonstrated in Table 10.

Table 9 Design example of soil deodorization facility

Facility to be deodorized: Sludge thickening facility
Deodorized air flow: 200 Nm^3/min.
Soil bed: 735 m^2
Void velocity: 4.5 mm/sec.

Table 10 Deodorization effect of soil odorization effect

Unit: ppm

Sample Name of matter	In let air	After soil deodorization (average at 4 places)
Ammonia	0.3	tr.
Trimethylamine	tr.	tr.
Hydrogen sulfide	0.44	0.0008
Methyl mercaptan	0.05	tr.
Methyl sulfide	0.01	tr.

The biological method can use composted sludge instead of soil, and its deodorizing peformance is as good as soil.

Another biological method is a simple one in which odors are blown into an aeration tank. Fig. 18 shows a flowchart of this simple method employed at the Fusekogawa Wastewater Treatment Plant, Sapporo.

There, foul air from the sludge processing systems such as sludge thickners is used as an air source for the aeration tank. The system shows a very high efficiency in removing malodorous substances, and the odors generated from the aeration tank are almost on a level with the system using fresh open air for aeration.

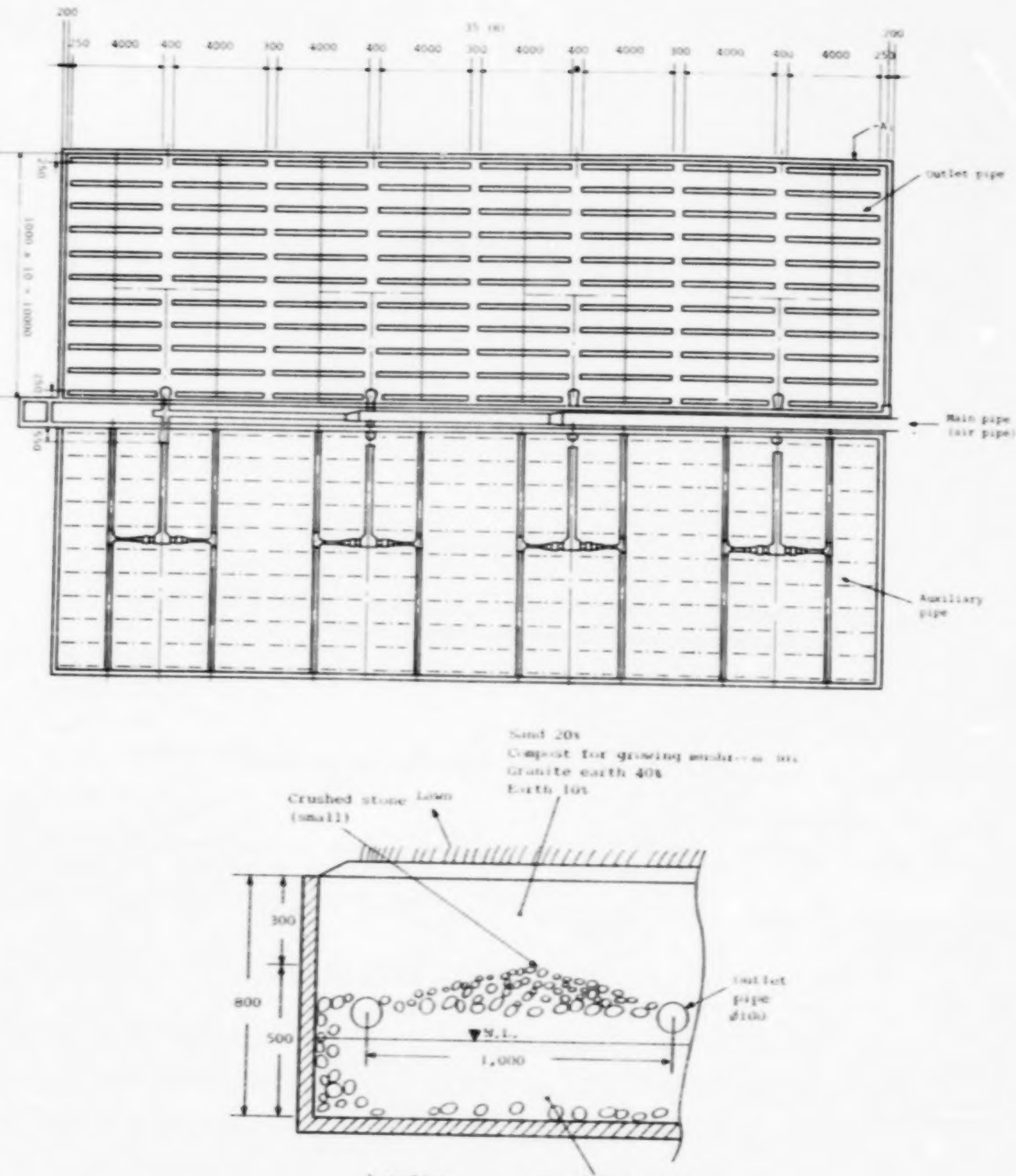
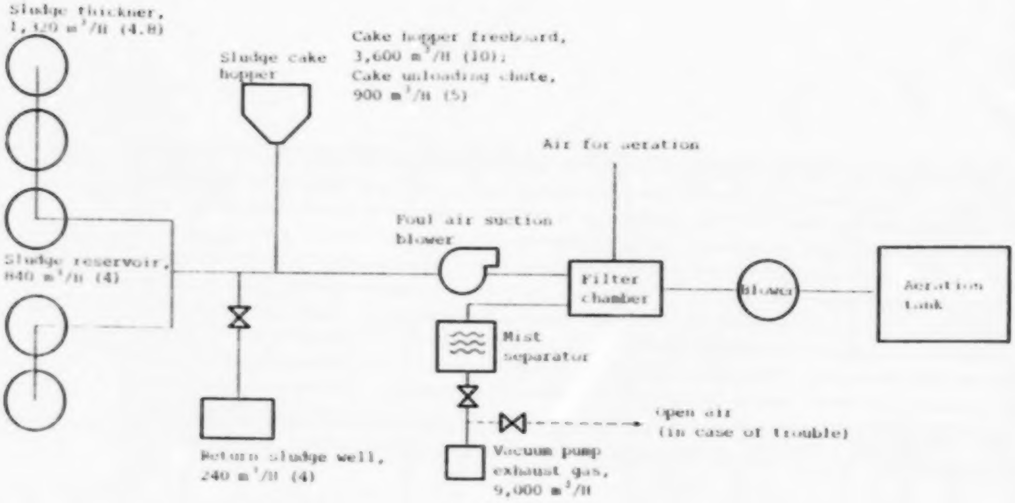

200
250
4000
400
4000
300
4000
400
4000
300
4000
400
4000
300
4000
400
4000
250
200
A
Outlet pipe
250
1000 × 10 = 10000
250
550
Main pipe (air pipe)
Auxiliary pipe
Sand 20%
Compost for growing mushrooms 30%
Granite earth 40%
Earth 10%
Crushed stone (small)
Lawn
300
800
500
Outlet pipe ⌀100
W.L.
1,000
A section
Gravel for bearing

Fig. 17 Construction of soil deodorization equipment


Sludge thickner,
1,320 m³/H (4.8)
Sludge cake hopper
Cake hopper freeboard,
3,600 m³/H (10);
Cake unloading chute,
900 m³/H (5)
Air for aeration
Foul air suction blower
Sludge reservoir,
840 m³/H (4)
Filter chamber
Blower
Aeration tank
Mist separator
Open air
(in case of trouble)
Return sludge well,
240 m³/H (4)
Vacuum pump exhaust gas,
9,000 m³/H

Blower capacity: 19,800 m^3/H

The values in m^3/H denote the foul air collecting rates.
The values in parentheses denote the number of ventilations per hour.

Fig. 18 Flowchart of deodorization system using aeration tank (Fusekogawa Wastewater Treatment Plant, Sapporo)

Table 11 Deodorization effect of aeration by activated sludge

Point of sampling		Threshold odor	Strength of odor	Unit: ppm						
				Odor type	Ammonia	Trimethyl-amine	Hydrogen sulfide	Methyl-mer-captan	Methyl sulfide	Carbon methyl
Entrance to wet filter room	Normal de-odori-zation	5,500	5	Ammonia, amine	66	0.651	0.53	0.54	0.24	0.170
Entrance to dry filter room		1,700	4	Rotten smell, ammonia	55	0.195	0.46	0.48	0.19	0.120
Entrance to aeration tank		55	3	Fusty smell	0.15	N.D. (<0.00005)	<0.0005	0.0064	0.018	0.0088
Mid-part of aeration tank		130	4	Fusty smell	0.32	N.D. (<0.00005)	<0.0003	0.0009	0.003	0.002
Exit from aeration tank		170	3	Fusty smell	0.11	N.D. (<0.00005)	<0.0002	0.0006	0.0012	0.0011
Entrance to aeration tank	Aera-tion by the open air only	730	4	Kitchen drainage smell	0.097	N.D. (<0.00005)	<0.0003	0.016	0.033	0.016
Mid-part of aeration tank		73	3	Sewer smell	0.18	N.D. (<0.00005)	<0.0002	0.001	0.0012	0.0028
Exit from aeration tank		73	3	Fusty smell	0.12	N.D. (<0.00005)	<0.0002	0.0011	0.0011	0.0014

The blower which had handled corrosive gases such as hydrogen sulfide for five years and six months was taken down for inspection, but neither pitting nor wear was noticed. The odor gas pipe needs drain wherever water is likely to be condensed because the odor gas contains much water. The metal fittings of the dry filter, gas flowmeter, etc. must be of stainless steel of protection from corrosion.

The replacement of dry filter element usually is once every five years, but its frequency must be increased to once every several months when handling odorous gases.

6. EXAMPLES OF DEORODIZATION MEASURES

Examples of 6 kinds of deodorization methods that have recently been used at the sewage treatment plant are explained here.

Table 12 Example of deodorization facility

Deodorization process	Active carbon absorption	Acid cleaning + alkali hypochlorous acid cleaning + active carbon	Special active carbon (neutral + acidic + alkali)	Catalytic reduction using ammonia	Soil deodorization	Oxidation using activated sludge
Facility to be deodorized	Aeration tank	Sludge thickner dewatering equipment	Sludge dewatering equipment	Sludge incineration	Sludge thickner	Sludge thickner dewatering
Capacity of facility (Nm^3/m)	1,000	350	395	1,800	1,978	265
Deodorization effect	85% or more	99%	96%	98% or more	92% or more	96% or more
Construction cost (thousand yen)	210,000 (cost in 1978)	206,000	-	410,000 (cost in 1983)	30,600 (cost in 1977)	-
Maintenance and administrative cost (thousand yen/year)	44,000	107,300	22,600	83,000	14,800	-
Construction cost per 1 Nm^3 (thousand yen)	210	589	-	228	15.5	-
Maintenance and administrative cost per 1 Nm^3 (thousand yen)	44	30.7	57.2	46.1	7.48	-

Table 12 is the summary of the object for deodorization, the capacity of facility, effect of deorodization, construction cost and operation and maintenance costs. Activated carbon is used for deodorization of the aeration tank, and the odor has characteristics that it has a large air flow but has low odor concentration. Therefore, it is possible to absorb the odor only by making the air pass through the absorption tower, and life of the activated carbon is long.

The catalytic reduction process is the method that removes nitrogen monoxide and nitrogen dioxide in the discharge gas from sludge incinerator.

Acid cleaning + alkali hypochlorous acid cleaning + active carbon has a good results of deodorization of the high concentration odor from the sewage treatment plant, but construction cost and maintenance and administrative costs in this method are very high. In the operation and maintenance costs, power consumption and chemical costs are 56% and 36% respectively and the rest is for regeneration of the activated carbon and water charges.

Operation and maintenance cost for special activated carbon is high and 90% of the cost is for regeneration of the special activated carbon. In this case, the activated carbon is regenerated once a year, but since it is possible to operate for almost 2 years without regeneration, the cost could be reduced to half of the cost shown.

In the case of soil deodorization, construction cost and operation and maintenance costs are rather low although it is effective. In this facility, aeration tank is installed in the building and the roof is used for deodorization, so land cost and building cost are not included in the cost shown in the Table. For instance, the site area is 3750 m^3 and if it is possible to secure the site, the construction cost would be less than other methods.

The aeration process by activated sludge is to modify the ventilation of the ordinary sewage treatment plant, so accurate construction cost and operation and maintenance cost are not obtained. But it is certain that this method is less expensive compared with other methods.

Reference

1) S. Kyosai, "Study on Desorption of Volatile Compounds Dissolved in Water", Japan Journal of Water Pollution Research, Vol. 4, No. 4, 1981 (in Japanese)

Tenth United States/Japan Conference
on Sewage Treatment Technology

SLUDGE DISPOSAL BY COMPOSTING
IN FUKUOKA

Yukio Hirayama
Director of Planning Division
Sewage Works Bureau
City of Fukuoka

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1 INTRODUCTION

As more and more areas are served by sewer systems, the amount of sewage sludge produced in Japan is increasing year by year. However, during the past several years sludge to be carried off for ultimate disposal has remained at almost the same level. This is due to the introduction of high-performance dehydrators capable of removing far more moisture than ever, the progress made in composting, and more dependance on incineration for sludge disposal.

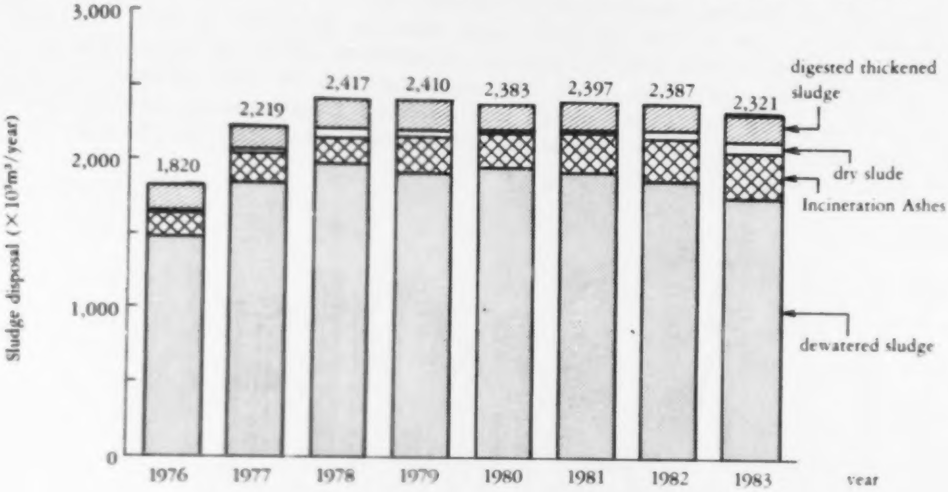


Figure 1. Sludge disposal 1976-1983 (nationwide) broken down by the form of sludge.

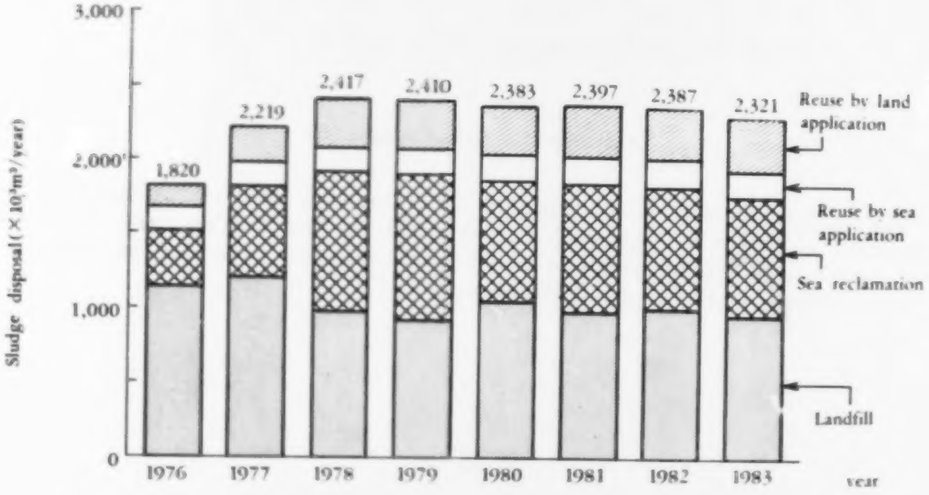


Figure 2. Sludge disposal 1976-1983 (nationwide) broken down by the type of disposal.

Up until recently, most sludge in Japan was landfilled. However, over the past several years it has become increasingly difficult to find sites for landfill. At the same time, voices calling for effective use of resources have been mounting. These two factors have prompted local authorities to study various ways of turning sludge into something reusable - building material, compost, etc. When the results of the studies showed that production of compost required less energy than that of building material, emphasis has been placed on the development of composting technology. The progress made in this field has now brought the number of municipalities which embarked on composting to approximately 40.

Fukuoka City took a lead in this move when it decided in 1967 to launch a "Sewage for Agricultural Use Project" - a project aimed at recycling sewage waste. In 1981, the City completed the construction of the "Sludge Control Center," a composting facility, which has enabled a systematic implementation of this recycling project. At present, all sludge produced in the City is recycled into compost.

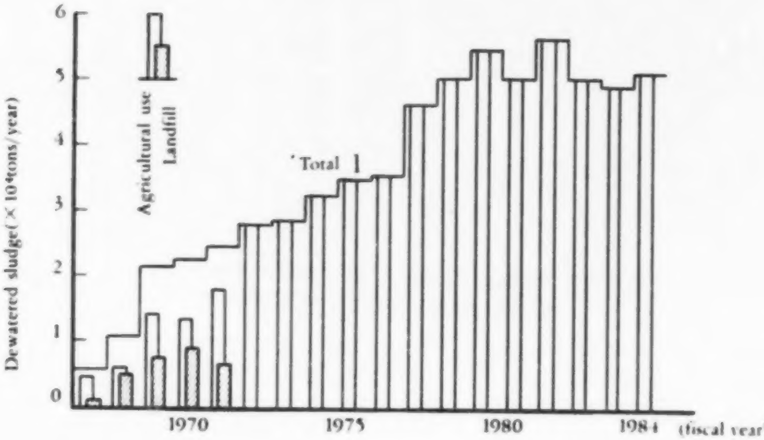


Figure 3. Sludge disposal in Fukuoka City.

In the following pages are reported the details of the "Sewage for Agricultural Use Project," general information on the Sludge Control Center including operational data and the quality of compost produced in this Center, and future problems of composting.

2 OUTLINE OF SEWERAGE SERVICE

2.1 GENERAL VIEW OF FUKUOKA CITY

Fukuoka City is situated in the southwestern part of the Japanese Archipelago, some 1100km west of the nation's capital of Tokyo. The City, bounded by Hakata Bay on the north and mountains on the other sides, is full of scenic places. Being close to the Chinese Continent, Fukuoka has been a gateway to foreign politics, economies, and cultures since 2,000 years ago.

Fukuoka became a city in 1889 with a population of 50,000. Now it has developed into the 8th largest city of Japan with 1.15 million inhabitants, playing an important role both in the economy and in the politics of Japan. It has a total area of 335.6km^2, 60% of which is covered with plains. The population density in the urban areas is 80 persons per hectare.

To promote internationalization of the City, Fukuoka went into a sister-city-relationship with Oakland, U.S.A. in 1962. It is now affiliated with two more major cities of the world - Kuangchou in China and Bordeaux in France. These affiliations have been greatly helping the citizens living in different countries to exchange ideas and information in wide-ranging fields.

2.2 SEWER SYSTEM OF FUKUOKA CITY

Fukuoka City launched its first sewerage project in 1930. The project in those days dealt only with domestic sewage water and rainwater. It was in 1963 when the first 5-years sewerage improvement project started that the construction of a treatment plant came to be included in the sewerage planning. Since then, efficiently coping with the expansion of urbanized areas, the City has been carrying out well-planned projects to improve its sewer system. With approximately 40 billion yen allocated annually for building related facilities and equipments, sewerage improvement project is now one of the three major public works undertaken by the City.

Fukuoka City has planned to expand sewer-system-service areas to 14,504 ha. As of March, 1985, 7,682 ha. or approximately 680,000 inhabitants which account for 59% of the total population have received the benefit of this sewer system. (See TABLE 1, 2 and Figure 4)

TABLE 1. Sewerage service plane of Fukuoka City

Category / Sanitation district (plant)		Saito-zaki	Wajiro	Tobu	Chubu	Nagao	Nanbu	Seibu	Nokata	Total
Type of system		Separate, Combined	Separate	Separate, Combined	Separate, Combined	Separate	Separate	Separate, Combined	Separate	—
Start of operation		1962	1975	1975	1966	1965	1975	1980	1975	—
As of March 1985	Service area	120	462	1,736	3,177	76	1,569	474	48	7,682
	Population in service area	4,900	30,600	111,600	337,200	11,700	135,500	40,500	8,400	680,400
	Treatment capacity (m³ day at maximum)	6,500	20,000	83,000	350,000	5,500	(80,000)	37,500	4,600	507,100
Total plan	Service area	153	1,270	3,935	2,697	227	3,320	5,417		17,019
	Population in service area	10,000	93,000	234,000	276,000	24,000	279,000	491,000		1,407,000
	Sewage (m³/day at maximum)	13,000	64,000	200,000	350,000	5,500	(429,376)	300,000		932,500
Remarks							1)		2)	

1)Sewerage area controlled by Fukuoka prefecture
2)Will be included in seibu treatment plant

TABLE 2. Sewerage service of Fukuoka City (As of March1985)

Total area of the city	33,564ha
Total area of urbanized districts	14,500ha
Total population	1,152,300
Total sewerage service area	7,682ha
Sewerage-service area in urbanized districts	53.0%
Population sewerage-servise area	680,400
Percentage of population sewerage-service area to total population	59.0%
Sewers lehgth	2,194km
Treatment capacities(of seven plants in total)	507,100m³/day

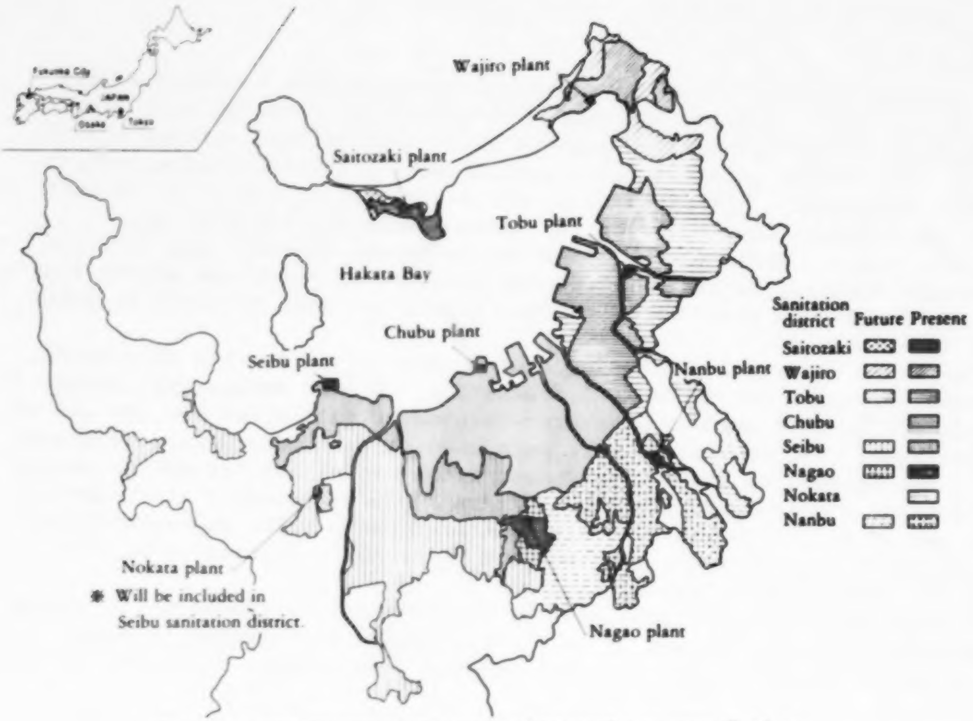

Fukuoka City
JAPAN
Osaka
Tokyo
Wajiro plant
Saitozaki plant
Tobu plant
Hakata Bay
Chubu plant
Seibu plant
Nanbu plant
Nokata plant
* Will be included in Seibu sanitation district.
Nagao plant
Sanitation district
Future
Present
Saitozaki
Wajiro
Tobu
Chubu
Seibu
Nagao
Nokata
Nanbu

Figure 4. Location map of treatment plants and sewerage districts.

The long-term plan calls on the City to extend this service to all of the urbanized areas by 1995. When this is realized, the system will serve 97% of the City's total population.

3 OUTLINE OF SLUDGE TREATMENT

At present, there are seven sewage treatment plants in Fukuoka City. These plants, when combined together, are capable of treating approximately 507,000m^3 of sewage per day (at maximum daily capacities), serving 680,000 persons. All sewage water is given conventional activated sludge process, secondary treatment. The effluent is all discharged directly, or through rivers, into Hakata Bay. Five of the seven sewage treatment plants - Saitozaki, Wajiro, Tobu, Chubu, and Seibu Treatment Plants - are equipped with sludge treating facilities. Sludge produced at Nagao and Nokata Plants which are smaller in size than the other plants are sent to Chubu or Seibu Plants for treatment.

The basic process for treating sludge consists of gravity thickening, anaerobic digestion and mechanical dewatering. Direct dewatering process is also applied to some extent. Dehydrators used at each plant now are all of pressure type. Vacuum filters were once used at the Chubu Sewage Treatment Plant. But because of its improved dewatering capability brought by recent technological development, pressure filters replaced vacuum filters and are now in use at all of the five plants mentioned above. The pressure filters have reduced sludge moisture content from 56 to 65% - figures which fall within the City's target range.

Sludge produced in fiscal 1984 was approximately 51,400 tons in volume with average moisture content of 64%. This figure is expected to double in 1995 to about 103,300 as the sewerage-service areas expand from year to year.

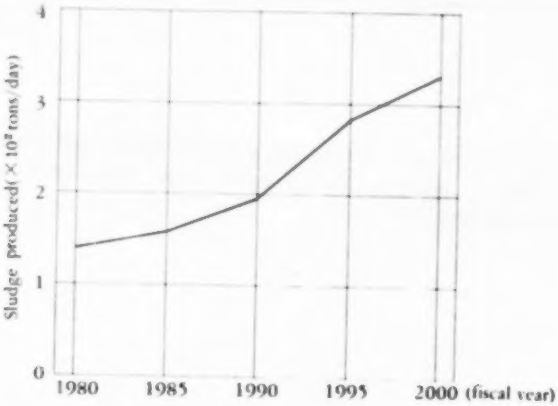


Figure 5. Forecast of sludge to be produced.

The following paragraphs explain the problems facing the Chubu and Tobu Treatment Plants. They are the major treatment plants of Fukuoka City, the Chubu Plant having maximum daily capacity of 350,000m^3 (main system, combined system) and the Tobu Plant 83,000m^3 (main system, separate system).

During the past decade ignition loss content of sludge at the Chubu Treatment Plant and the Tobu Treatment Plant has risen by 27% and 37% respectively. This increase is attributable to people's diversified diet and expansion of separate system areas, which in turn have brought a change in substrate that flows off to the plants in sewers. In addition, solids content has decreased from 4% to 3% at the Chubu Plant and from 4% to 2.5% at the Tobu Plant.

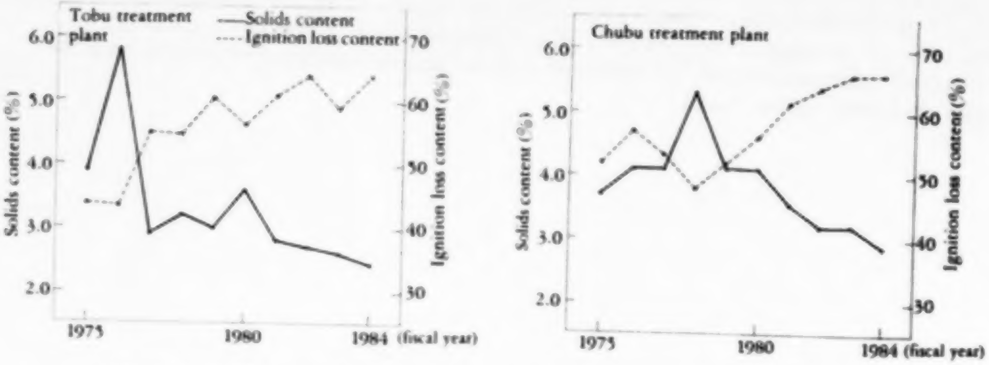


Figure 6. Solids content and ignition loss of thickened sludge 1975~1984.

This has resulted in decline of sludge thickening, and has been affecting treatment process at various stages: the gravity system thickening tanks now work only as storage tanks; increase in the amount of sludge carried into the digestion tanks causes insufficient digestion due to the shortage of stay; degrading of digestion supernatant liquor has increased return water load. More water in sludge has also increased the use of dewatering auxiliaries. During the past five years at the Chubu Treatment Plant, for example, 30 to 50% more dewatering auxiliaries had to be added to the sludge to obtain a given moisture content.

TABLE 3. Information on dewatered sludge

(1984 fiscal year)

Category \ Treatment plant		Saitozaki	wajiro	Tobu	Chubu		Seibu
Slndge treatment process		Thickening + Anaerobic digestion	Thickening + Anaerobic digestion	Thickening + Anaerobic digestion	Thickening + Anaerobic digestion	Thickening	Thickening
Dehydrator		Filter press	Filter press	Filter press	Vacuum filter	Filter press	Belt press
Coagulant		Lime + Ferric chloride	Lime + Ferric chloride	Lime + Ferric chloride	Lime + Ferric chloride	Lime + Ferric chloride	Lime + Ferric chloride
Moisture content (%)	Range	57.1~66.2	52.6~68.6	58.4~66.1	69.1~75.0	48.1~62.9	61.1~74.7
	Average	61.1	64.8	61.6	72.1	56.1	70.3
Ignition loss content (%)	Range	40~52	33~55	37~50	29~38	28~46	37~57
	Average	46	44	42	34	35	47

Increase of inorganic matter in dehydrated sludge may cause another problem in sewage disposal, as it raises pH value.

To cope with these problems, Fukuoka City set up picket fences in the thickening tanks and installed vacuum protection devices (an equipment to improve dehydration rate by covering the top of the drum with a vinyl sheet) to the vacuum filters. However, more important than anything else is higher thickness of sludge. Therefore, discussions on the introduction of mechanical concentration or pressure floatation concentration of excess sludge are now under way.

4 COMPOSTING OF SEWAGE SLUDGE

4.1 MOVE TOWARD AGRICULTURAL USE

4.1.1 DECISION MAKING

In regards to disposal of sludge, since the time the Chubu sewage disposal plant commenced operation adopting the conventional activated sludge method in 1966, we have been consistently restoring it for the creation of green zones and agricultural areas.

In starting sludge disposal, Fukuoka City discussed such ways of disposing the most of dewatered sludge as landfill or agricultural use.

The results of discussion are as below:

(1) Utility of sludge as resources

Consisting of nitrogen, phosphorus, organic substances, and minerals of various kinds, sludge can make good fertilizer.

(2) Nontoxicity

Heavy metal content in sludge proved to be considerably low. The reason for this is that Fukuoka is a commercial city with most citizens engaged in tertiary industry and that consequently the City's sewage is primarily domestic waste water. Also, waste water from industrial operations mainly comes from food processing industry.

TABLE 4. Heavy metal content in sewage sludge

Heavy metal	Fukuoka City	Japan	U.K.	U.S.A.	W. Germany	Sweden
Zn	720	1,760	1,820	1,700	2,000	1,567
Cu	170	350	613	800	400	560
Ni	22	80	188	80	—	51
Cr	20	—	744	500	—	86
Cd	1.5	3.7	29	10	10	6.7
Pb	26	62	550	500	400	180

1) Average values in 26 different sludges produced in September, 1979 ; Japan Soil Association
2) Average values in sludges disposed of on land in 1977
3) Medium values in typical sludges by R.L. Chany
4) Heavy metal in sludges handled by Fedaral Water Bureau : Lowest limits of interim guide values by U. Mällen
5) Beggren and Oden report(1972); Medium values shown in data at 93 treatment plants

(3) Availability of land to fertilize

There still remained a large area of land fertilize in and around the central part of the City.

(4) Future outlook for sludge disposal

If the City was to continue to dispose of sludge into landfills, the problem about the availability of land to fill was expected to arise.

Since the start of the project, with the improvement made in the composting technology (a shift from sludge drying process to high-rate composting process), the products have gained better qualities and become easier to handle. This has boosted the demand for the products to the extent that all dewatered sludge produced in the City can be used at fields.

However, the way to composting was not without any obstacle, because people complained about odors emitted from compost manufacturing factories. In the wake of this public nuisance a project to discuss future measures for sludge treatment was launched in 1975.

This project deals with:

(1) Making an estimate of future increase of sludge

(2) Making an estimate of agricultural area which needs to be supplied with compost

(3) Discussing systematic way of sludge recycling

(4) Making research on composting technology

(5) Making plans on experimental use of compost to test its effects on soil and crops

In addition to this project, the construction of the Sludge Control Center, a composting facility, was started in 1979 and completed in 1981. The completion of this facility has precipitated the implementation of the "Sewage for Agricultural Use Project".

4.1.2 RECYCLING SYSTEM

Compost made from sludge must meet the requirements below to ensure effective recycling.

(1) Must be of good quality.

(2) Stable supply and proper prices must be maintained.

(3) Voices of the users must be reflected in the products.

(4) Proper instructions must be given as to the use of compost according to soil and crops to grow.

(5) Inspection on the quality of compost and on the soil after the supply of compost must be carried out in an efficient way.

(6) Effort must be made to tap the market for compost by expanding its sales network.

Fukuoka City deliberated on these requirements and concluded that joint efforts by both the municipality and citizens were indispensable for the promotion of this recycling project. Figure 7 shows how the project works in collaboration with the private sector.

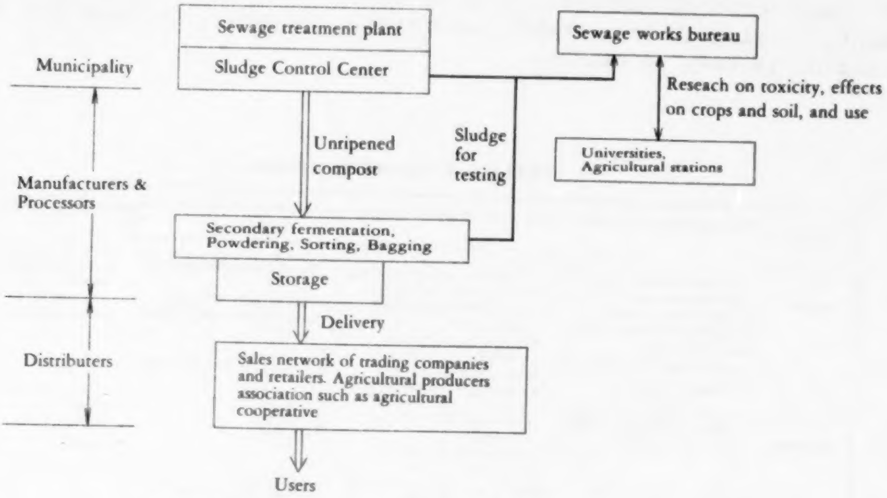

Sewage treatment plant
Sludge Control Center
Sewage works bureau
Municipality
Reseach on toxicity, effects on crops and soil, and use
Unripened compost
Sludge for testing
Universities, Agricultural stations
Manufacturers & Processors
Secondary fermentation, Powdering, Sorting, Bagging
Storage
Delivery
Distributers
Sales network of trading companies and retailers. Agricultural producers association such as agricultural cooperative
Users

Figure 7. Recycling system for agricultural use.

4.2 SLUDGE CONTROL CENTER

4.2.1 CONSTRUCTION PLANNING

Basic requirements of the composting facility were set as follows:

(1)	Preparatory step:	"Non-Adding Method"
(2)	Sludge to be handled:	Pressure dewatered sludge & vacuum dewatered sludge
(3)	Moisture content after fermentation:	Less than 35%

The reasons for setting the above were:

1. Preparatory step before fermentation includes moisture and pH adjustment, improvement of permeability, and addition of seed microbe. In general, there are two different ways of preparation. One is "Adding Method Preparation" in which organic materials are added to the sludge; the other is "Non-Adding Method Preparation" which returns fermented sludge to circulate. Organic materials such as rice husks, straw, sawdusts, or wood chips to be added to the sludge are difficult to obtain in sufficient quantities. Preservation also causes a problem. In addition, high transportation costs and longer ripening period gave way to the "Non-Adding Method Preparation".

2. Sludge dewatered with polymer coagulant was not taken for experiments because of its lower permeability. Instead, sludge treated with lime of ferric chloride as dewatering aid (pressure dewatered sludge and vacuum dewatered sludge) was selected.

3. Ultimate moisture content of 30 to 35% does not allow compost to referment, putting it in a stable condition. Compost with this much moisture content is easy to handle.

TABLE 5. Composting process

Process	Dewatered sludge → Preparatory step	→ Primary fermentation → ← Return compost ←	Secondary fermentation (Ripening) →	Powdering, Graining, Sorting
Purpose	To creat a better environment for microorganism to grow * Improvement of permeability * Adjustment of moisture content * Adjustment of pH * Adjustment of nourishment • Addition of seed microbe	To perform aerobic fermentatinn * Decomposition and stabilization of easily decomposable organic matter * Destruction of germs and parasites * Inactivation of weed seeds * Lowering of moisture content	To perform aerobic fermentation * Decomposition and stabilization of decomposable organi matter * Production of humic substance	To add more value to the product. (To make the product easier to hanhle)
Operation	Mixing of dewatered sludge and return compost	* Air supply by blowers * Turning * production of return compost	* Turning * Air supply	* Sieving * Graining * Bagging
Remarks	* In some cases, bulking agent replace return compost. * In some cases, neither bulking agent nor return compost is added.	* In some cases, compost is not returned.	* In some cases, air is not supplied. * In some cases, primary and secondary process are performed in the same tank.	

The experiments based on the above conditions showed the following results:

(1) Ratio of dewatered sludge to compost

When using vacuum dewatered sludge, the moisture content of the feed mixture (a mixture of vacuum dewatered sludge and return compost) should not exceed 50%; turning operation for permeability would otherwise be affected. If the moisture content rises above 55%, sludge becomes anaerobic resulting in unstable fermentation.
If more return compost is added to the feed mixture, it can reduce the moisture content, but will also reduce the amount of easily decomposable organic matter in the mixture shortening the high-heat lasting hours.
To keep fermentation in process, moisture content of 45% or more is necessary. The results of the experiments reveal that the optimum moisture content of the feed mixture is 47 to 48%. (for a mixture in which pressure dewatered sludge and vacuum dewatered sludge are mixed at the ratio of 1 : 1 in terms of wet weight.)
The weight ratios of sludge to compost should be 1 : 2.0 to 3.0 for vacuum dewatered sludge and 1 : 1 for pressure dewatered sludge for stable fermentation.

(2) pH conditioning

pH is closely related to the number of days required for fermentation. To facilitate fermentation, pH of 10 or below is desired. Sludge dewatered with the aid of lime shows pH value of about 12. Adding return compost to this sludge can reduce the pH value of the mixture to 11. In addition, carbon dioxide in the gas generated during fermentation works as a neutrilizer when this gas is circulated for air supply after being dehumidified. As a result pH value is further decreased to 9.5, the optimum pH for fermentation. The pH value after fermentation is completed will be less than 8.

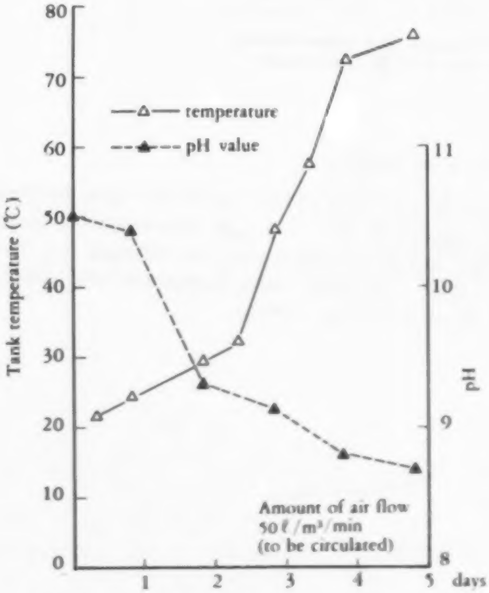


Figure 8. Variation of fermentation tank temperature and pH value.

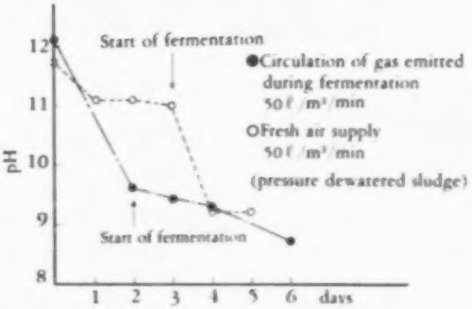


Figure 9. Variation of pH value.

(3) Amount of air flow and turning operation

The optimum amount of air flow varies depending on environmental conditions such as atmospheric heat or humidity, and on the condition of sludge. Until the fermentation temperature rises high enough, 60-100 litters of air per minute per cubic meter of the mixture, and during and after the water sprinkling period that follows, 150 litters of air per minute per cubic meter of the mixture is required to expedite fermentation.

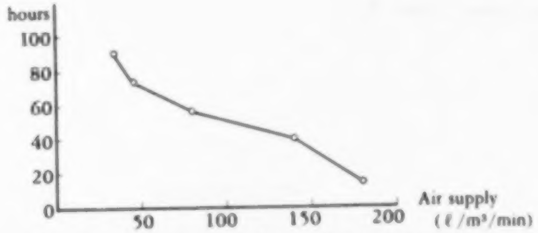


Figure 10. Amount of air supply and hours during which temperature of 65°C or above is maintained.

(4) The number of days required for fermentation

Fermentation process is controlled with attention paid to the fermentation temperature, heu, odors, and density of the gas produced. Seven to ten days is required before the moisture content decreases to less than 40%, and 10-14 days to less than 35%. If the room temperature lowers to 15°C or below, two more days or so should be added.

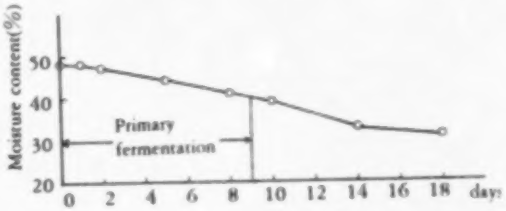


Figure 11. Variation of moisture content.

a) Selection of Fermentation Tank

The fermentation tank must be designed as to meet the needs below:

(1) How far the fermentation has proceeded in the tank must be checked easily.

(2) Structure must be simple and flexible to handle different qualities and quantities of sludge.

(3) Construction costs should be low.

(4) Deordorization must be simple and easy.

(5) Turning must be carried out to the full extent.

(6) Adjustment of air flow in accordance with the progress of fermentation must be made easily.

Based on these needs, study on the designs of various types of fermentation tanks was made. The results of the study revealed that, although the "plane fermentation type" took up more space and was more difficult to cope with odors, it had more merits in the other points. Return compost, on the other hand, can easily ferment if primary fermentation proceeds in good condition. Thus, we concluded to adopt "plane fermentation type" with vertical silo type which is efficient in heat insulation and therefore facilitate secondary fermentation. As for a tool for turning, shovel loaders were chosen.

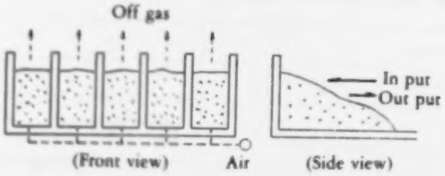


Figure 12. Plane type fermentation tanks.

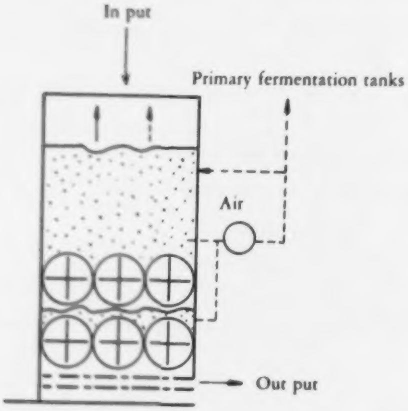


Figure 13. Vertical type fermentation tanks.

4.2.2 THE OUTLINE OF THE FACILITIES

The construction of the Sludge Control Center scheduled to have a composting capacity of 206 tons/day started in 1979. The construction cost was about 2.2 billion yen. The center was completed in 1981 and operation began in January, 1982.

a) Facilities

TABLE 6, Figure 14 and TABLE 7 show the outline of the facilities, the flow chart and the main equipment respectively. The facilities consist of (1) pre-adjustment equipment, (2) primary fermentation equipment, (3) return-compost production equipment, (4) deodorization equipment and others such as vehicle washing and waste-water treatment equipment. (See TABLE 6, 7 and Figure 14)

TABLE 6. General Information on the Sludge Control Center

<table>
<tr><td colspan="3">Site area</td><td>16,000m^2</td></tr>
<tr><td colspan="3">Floor area</td><td>7,800m^2</td></tr>
<tr><td colspan="3">Design volume of sludge for treatment</td><td>206t/day
Vacuum dewatered sludge 100t/day (Moisture content 75%)
Pressure dewatered sludge 106t/day(Moisture content 60%)</td></tr>
<tr><td colspan="3">Design production capacity</td><td>110t/day
Primary fermentation sluge(Moisture content 40%)</td></tr>
<tr><td colspan="3">Type of fermentation</td><td>"Non-Adding method" Plane fermentation</td></tr>
<tr><td colspan="3">Fermentation tank capacities</td><td>Vertical fermentation tank 117m^3 8 units
Plane fermentation tank 792m^3 13 units</td></tr>
<tr><td rowspan="6">Construction</td><td colspan="2">Construction period</td><td>1979-1981</td></tr>
<tr><td rowspan="5">Cost
(2247 million yen)</td><td>Site</td><td>156 million yen</td></tr>
<tr><td>Civilengineering and construction</td><td>879 million yen</td></tr>
<tr><td>Mechanical and electric equipment</td><td>1110 million yen</td></tr>
<tr><td>Site preparation</td><td>65 million yen</td></tr>
<tr><td>Designing and reasearches</td><td>37 million yen</td></tr>
<tr><td colspan="3">Start of operation</td><td>January, 1982</td></tr>
</table>

TABLE 7. Specifications of main facilities

Facility	Name	Specification	Unit
Material reservoir	Plane type tank	733m³(9 × 22 × 4m)	2
	Material sludge hopper	12m³	2
	Conveyor	450 × 36.0m	2
Return compost production	Steel plant vertical type tank	83m³(6.5 × 5.0 × 3.8m)	8
	Sludge hopper for return compost	4m³	1
	Conveyor	600 ~ 750 × 27.0m	7
	Blender	50m³/h	1
	Blower	ϕ400 × 103m³/min × 75KW	1
Primary fermentation	Plane type tank	733m³(9 × 22 × 4m)	11
	Return compost hopper	4m³	1
	Conveyor	450 ~ 1,100 × 30.0m	19
	Blender shredder	50m³/h	2
	Blower	ϕ500 × 175m³/min × 90KW	2
Deodorization	Acid cleaning · Alkali cleaning + Activated carbon absorption	570m³/min	1
Ventilation	Blower	ϕ1,050 × 780m³/min × 37KW	1
Wastewater treatment	Biological contact aeration	0.2m³/min	1

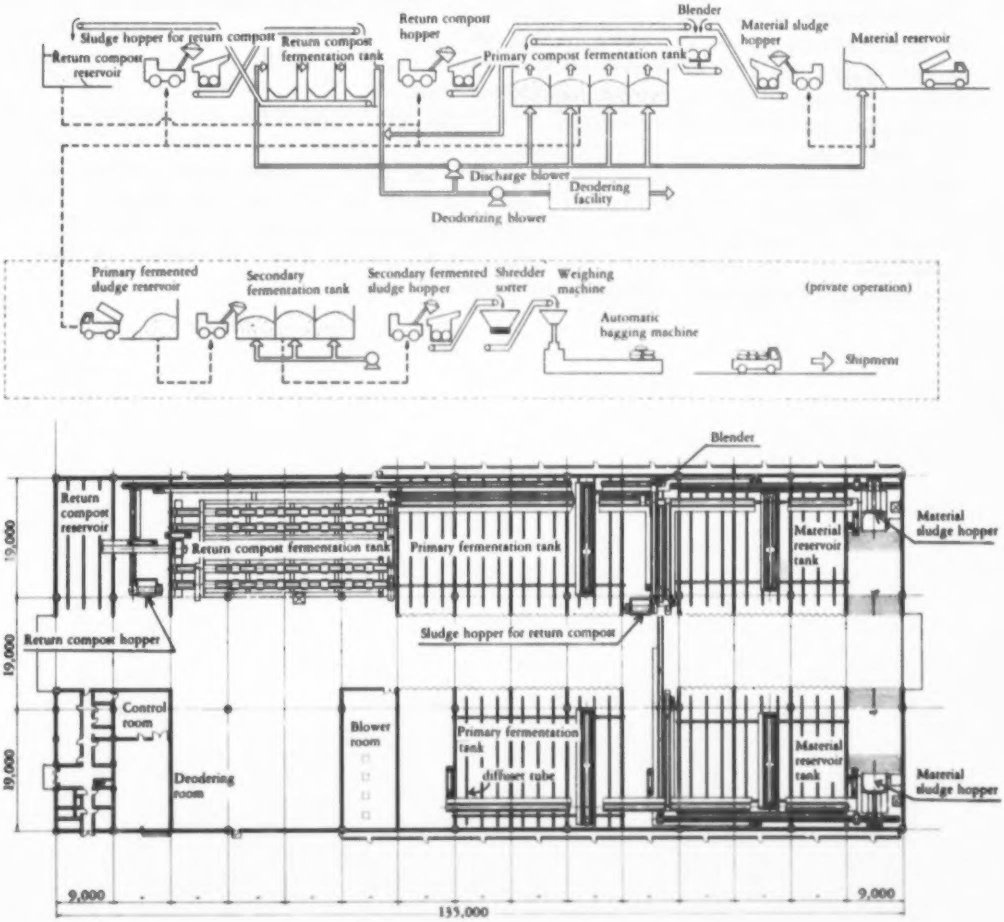

Sludge hopper for return compost
Return compost reservoir
Return compost fermentation tank
Return compost hopper
Primary compost fermentation tank
Blender
Material sludge hopper
Material reservoir
Discharge blower
Deodering facility
Deodorizing blower
Primary fermented sludge reservoir
Secondary fermentation tank
Secondary fermented sludge hopper
Shredder sorter
Weighing machine
Automatic bagging machine
(private operation)
Shipment
Blender
Return compost reservoir
Return compost fermentation tank
Primary fermentation tank
Material reservoir tank
Material sludge hopper
Return compost hopper
Sludge hopper for return compost
Control room
Deodering room
Blower room
Primary fermentation tank
diffuser tube
Material reservoir tank
Material sludge hopper
19,000
19,000
19,000
9,000
9,000
135,000

Figure 14. Compost production flow chart.

b) Operation Method

Dewatered sludge brought in from treatment plants is conveyed to the material reservoir where it is blended by tyre shovel to acquire uniform-quality. While it is stored, the material is aired with the waste gas from fermentation tanks to lower its pH value. Then the material is transferred into the blender-shredder to be blended-shredded with return compost. When this is completed, the mixture is conveyed to a fermentation tanks. This mixture is aerobic-fermented by being compulsively aired (50 - 100ℓ/misture m^3. minute) with the demoisturised waste gas supplied from the vertical-type fermentation tanks through three gas-discharge pipes equipped at the bottom of the tanks. The temperature of the mixture rises to its peak about one to three days after the mixture has been transferred to the fermentation tank. After the peak temperature (75° to 80°C in ordinary cases) is maintained for a certain time period (more than 24 hours), a large amount of air (100 to 150ℓ/mixture m^3.minute) is supplied so that the moisture of the mixture is discharged into the atmosphere and the temperature declines. When the temperature of the compost is lowered to 50° - 60°C, a tyre shovel turns this compost. The temperature rises again. Then, a part of the fermented material is conveyed into the vertical-type fermentation tank for the production of return compost. All remaining material is aired and turned twice or three times to be secondary-fermented. When all these processes are completed, the product is sorted and bagged. After the first transferal of the mixture, it takes seven to ten days to complete the primary fermentation and 14 to 16 days to complete all processes to obtain finished product.

4.2.3 THE STATE OF OPERATION

Since its start of the operation in January 1982, the facility has been operating successfully and treating all of the dewatered sludge produced in municipal treatment plants as originally scheduled. TABLES 8 and 9 show the results and the indicators of the operation.

TABLE 8. Operation results

(Unit : tons)

Item Fiscal year	Dewatered sludge	Primary fermentation sludge	Product compost
1983	49,298	24,522	14,488
1984	51,369	25,685	16,393

Note : Product compost does not include shipment in bulk.

TABLE 9. Operation indicators

Item	Indicator
Deodorizing	Little ammoniac odor. No putrid smell.
Resolving of organic substance easy to be resolved.	BOD less than solid substance 30 mg/g
Killing of pathogenic bacteria and parasite eggs. Deactivating weed seeds.	Fermentation temperature : over 65℃ for 48 hours.
Moisture content of the delivered primary fermented material	40 % and less
Hue	Reddish brown as a result of ferric chloride in the sewage sludge being oxidized
pH after fermentation	8.5 and less

a) Property Changes of Sludge

Property changes of sludge in May 1985 were as follows.

TABLE 10. Property analysis in each process

(1985.5.1～5.15)

	Analysis item	Moisture content	Ignition loss	pH	T-C	T-N	BOD_5
	Unit	%	%	(H_2O)	%	%	mg/g
Dewatered sludge (Five treatmen plants)	Average (max～min)	65.4 50.8～71.8	37.3 28.1～54.9	12.4 11.5～12.6	20.5 15.9～26.3	2.4 1.6～3.5	144 73～215
Return compost	Average (max～min)	34.0 30.8～34.8	24.9 24.1～26.2	8.2 8.1～ 8.3	16.6 14.2～17.6	1.3 1.2～1.3	10 9～ 12
Feed mixture	Average (max～min)	45.6 45.2～45.9	28.8 28.2～29.4	10.9 10.4～11.1	18.6 17.9～19.2	1.7 1.6～1.7	58 52～ 63
Primary fermentation sludge	Average (max～min)	39.0 37.0～42.4	25.2 24.9～25.3	8.3 8.2～ 8.4	17.3 16.9～17.6	1.3 1.2～1.4	13 12～ 14
Product compost	Average (max～min)	30.8 23.6～31.5	24.1 22.9～24.3	8.0 8.0～ 8.1	16.5 14.6～17.3	1.3 1.2～1.3	12 11～ 13

Notes : Ignition loss, T-C, and T-N refer to dry.

(Temperature)

As Figure 15 shows, the temperature reaches its peak of over 80°C, 60 to 70 hours after the first transfer of the mixture and remains at 65°C for 75 hours. When the temperature is maintained at over 65°C for 48 hours and more, almost all pahtogenic bacteria die and weed seeds are deactivated. (See Figure 15)

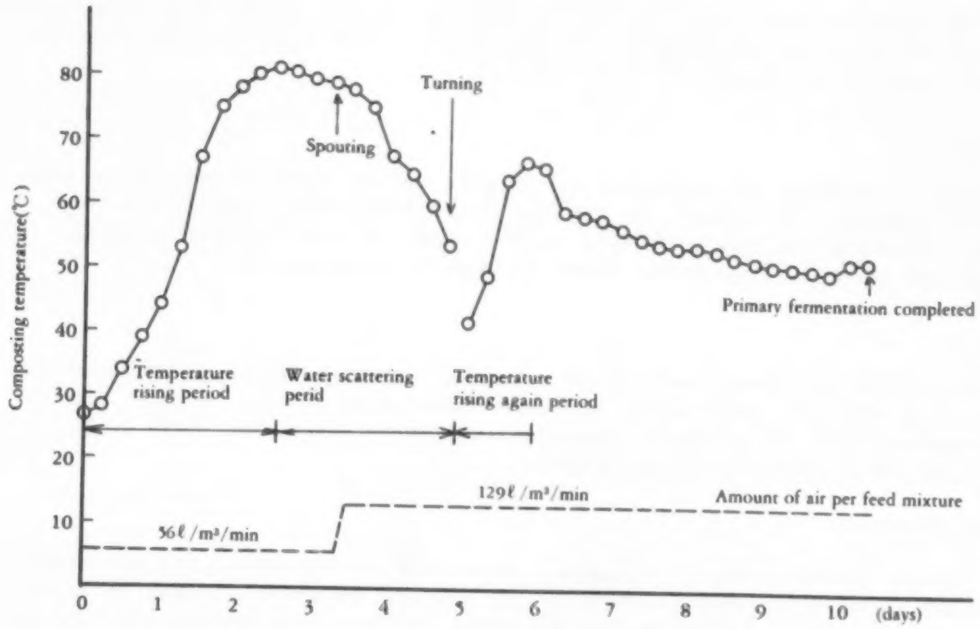


Figure 15. Change in composting temperature with time.

(Airing)

Amount of air for airing when the temparature is rising...
40 - 80ℓ/mixture m^3.minute

Amount of air for airing after the peak temperature is achieved...
100 - 150ℓ/mixture m^3.minute

Amount of air for airing of the vertical-type fermentation tank...
158 - 173ℓ/amount of the material put into the fermentation tank m^3. minute

(Moisture Content)

The moisture content is reduced to 47% after the dewatered sludge of the water content of 65% and the return compost of 37% are mixed. After the primary fermentation, it is further reduced to 39% and the ultimate product contains 31% moisture which shows favorable drying and fermentation conducted.

(Ignition Loss)

The ignition loss which is 28% on average at the stage of feed mixture declines to 25% after the primary fermentation. The figure of the finished product compost is similar to that and 24%.

(BOD_5)

BOD_5 is used as an indicator of organic matter easy to resolve. Solid material has BOD_5 of 144mg/g on average which reduces to 13mg/g after the primary fermentation and to 12mg/g at the stage of the finished product. BOD removal rate is 89%, which shows favorable fermentation.

(pH)

The pH of the dewatered sludge is about 12 which declines to about 11 at the feed mixture stage which has been mixed with return compost. After the completion of the primary fermentation, this figure goes down to 8.3 and to 8.0 at the finished product stage.

(Weight Changes)

Figure 16 shows weight changes. About 60% or 40tons of the moisture contained in material sludge (65tons per 100tons material sludge) is volatilized during primary fermentation and about 10% or 5tons during the process of producing return compost. The moisture content of the sludge to be sent to secondary fermentation is about 30% or 20tons of the original 65tons. During secondary fermentation, 7tons of moisture is further volatilized. This means that composting helps volatilization of 80% of the material sludge. Fifty percent of ignition loss is also observed.

As a result, the weight of material sludge is reduced to 50% after the primary fermentation and to about 42% after the secondary fermentation. (See Figure 16)

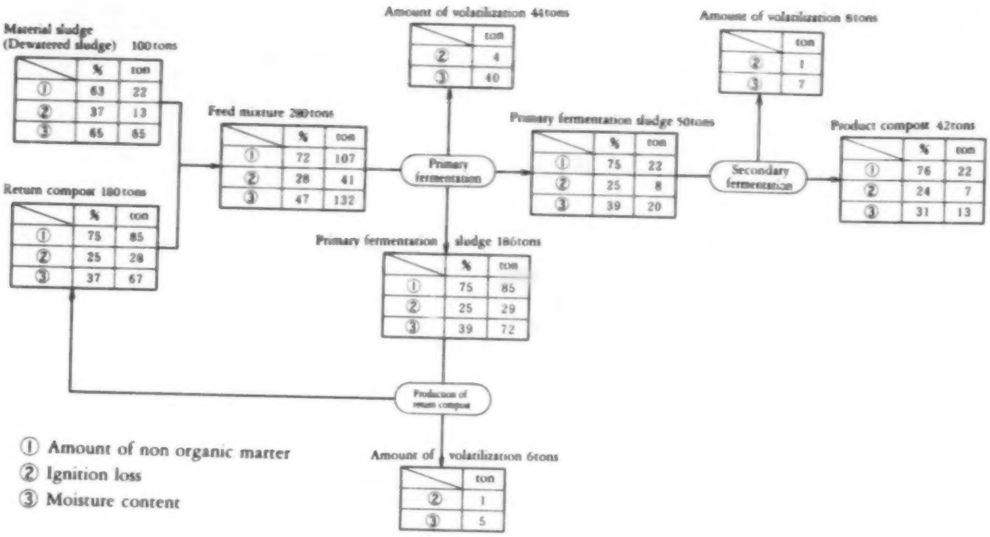

Material sludge
(Dewatered sludge) 100tons
% ton
① 63 22
② 37 13
③ 65 65
Return compost 180tons
% ton
① 75 85
② 25 28
③ 37 67
Feed mixture 280tons
% ton
① 72 107
② 28 41
③ 47 132
Amount of volatilization 44tons
ton
② 4
③ 40
Primary fermentation
Primary fermentation sludge 50tons
% ton
① 75 22
② 25 8
③ 39 20
Amount of volatilization 8tons
ton
② 1
③ 7
Secondary fermentation
Product compost 42tons
% ton
① 76 22
② 24 7
③ 31 13
Primary fermentation sludge 186tons
% ton
① 75 85
② 25 29
③ 39 72
Production of return compost
Amount of volatilization 6tons
ton
② 1
③ 5
① Amount of non organic matter
② Ignition loss
③ Moisture content

Figure 13. Daily mean mass balance.
(per 100 tons of material sludge)

b) Fertilizing Elements

Concerning quality control of the product. property analysis is conducted twice a month to ascertain the fertilizing elements of the compost. Table 11 shows the result of property analysis in 1984. The compost is a slow-effect fertilizer and is abundant in nitrogen and phosphate even though potassium content is low. The product is favorably evaluated as being able: (1) to bring about harvest increase, (2) to have a restricting effect on growth retardation by continuous cropping and on soil pathogenic bacteria, (3) to improve quality and (4) to have soil ameliolation. (See TABLE 11)

TABLE 11. Constitution of sludge compost

Item	Constituent amount
Moisture content(%)	30.8
T-N (%)	1.3
T-P_2O_5(%)	2.8
T-K_2O(%)	0.2
T-C (%)	16.5
C/N	12.7
Ignition loss(%)	24.1
pH(H_2O)	8.0

Notes: T-N, T-P_2O_5, T-K_2O, T-C, and Ignition loss refer to dry compost.

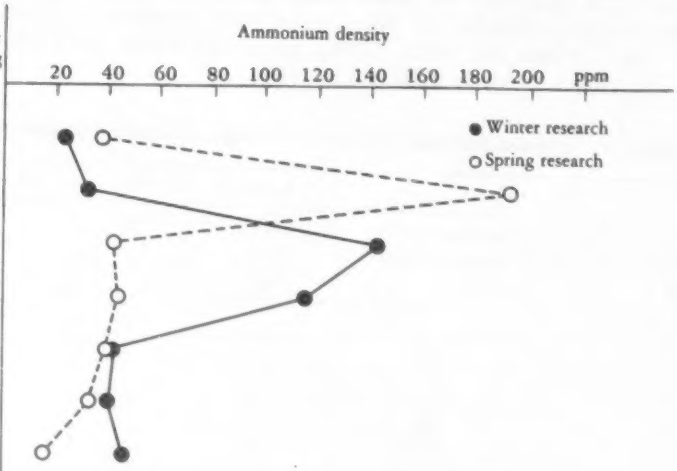
c) Odors

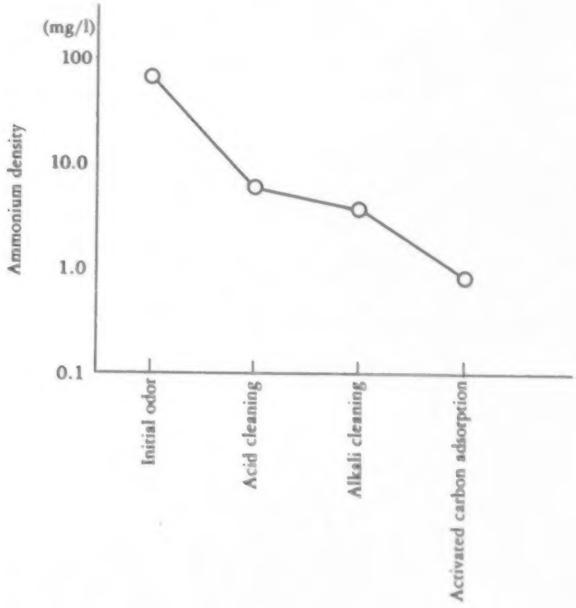
Odors in fermentation tanks have a high percentage of ammonium and other sulfides such as methyl mercaptan, methyl sulfide and methyl bisulfide. The ammonium density declines as fermentation progresses but when the amount of air for airing increases or turning takes place it increases again. In this facility's case, the odor is removed by sucking blowers from the fermentation tanks, then acid cleaning and alkali cleaning is conducted. After the odor level is thus reduced, the activated-carbon absorption treatment is applied to acquire favorable conditions regarding odor treatment. (See TABLE 12)

TABLE 12. Odor matter density.

		Methyl mercaptan	Hydrogen sulphide	Methyl sulphide	Methyl bi-sulphide	Trimethyl amine	Acetaldehyde	Styrene	Research date
Odor density mg/l		0.34	ND	0.18	0.60	ND	ND	ND	Winter 18 January
		0.01	ND	0.01	0.03	ND	ND	ND	Spring 18 March

Day passed after fermentation	Airing amount Winter	Spring
1	60	55
2	59	72
3	84	96
4	84	95
5	71	94
6	71	90
7	84	88
days	ℓ/mixture·m³·min	





4.2.4 UTILITIES COSTS

TABLE 13 shows the results of calculation of utilities costs per one ton of dewatered-sludge dry material treated at the facility. Calculation factors are the amount of dewatered sludge, required electricity and chemicals used. The utilities cost per one ton is about 4,200 yen. The electric power cost for as much as about 87% of the cost. When the personnel cost needed for the operation of the center is taken into consideration, the cost is about 7,100 yen per one ton of dewatered-sludge dry material. On the other hand, the calculation result of the costs when sludge is incinerated is about 11,300 yen. Compared with this, the utilities cost of the Sludge Control Center is lower. (See TABLE 13, 14)

TABLE 13. Utilities cost (Sludge Control Center) per one ton of dewatered sludge material

		Amount	Unit	Unit(yen)	Cost(yen)
Electricity	High voltage equipment (400V)	134.6	KWh	21.7	2,921
	Low voltage equipment (200V)	21.9	〃	21.7	475
	Lighting	10.1	〃	21.7	219
Fuel	A heavy oil	3.3	ℓ	64.0	211
Water	Water supply	0.7	m^3	303.7	213
Chemicals (Deodorizer)	Hydrochloride (35%)	2.1	kg	27.7	58
	Sodium hydroxide (20%)	0.7	〃	18.3	13
	Activated carbon	0.05	〃	620.0	31
Water treatment	Sludge treatment	0.006	m^3	2,390.0	14
	Tablet of chlorine	0.002	kg	540.0	1
Others	Chart sheets	0.002	Bundles	1,000.0	2
Total					4,158
Personnel cost	Equipment mentenance	0.14	person	12,300	1,722
	Shovel operation	0.05	〃	12,100	605
	Person on night duty	0.02	〃	8,900	178
	Cleaner	0.07	〃	6,500	455
Total					2,960
Grand total Per one ton of dewatered sludge (Moisture content 65%)					7,118 (2,491)

TABLE 14. Utilities cost required for incinerating one ton of solid sludge material

	Unit price	Standard flow			Standard flow with incineration deodorize		
		Vertical multistage incinerator	fluidized-bed incinerator	Rotary drying oven	Vertical multistage incinerator	fluidized-bed incinerator	Rotary drying oven
Fuel	ℓ	—	120	80	—	120	80
A heavy oil	64yen/ℓ	—	7,680	5,120	—	7,680	5,120
Fuel	ℓ	—	—	—	285	—	240
Kerosine	67yen/ℓ	—	—	—	19,095	—	16,080
Electricity	KWH	300	330	320	300	330	320
	21.7yen/KWH	6,510	7,161	6,944	6,510	7,161	6,944
Chemicals	kg	10	20	20	10	20	20
	41yen/kg	410	820	820	410	820	820
water	m^3	90	95	145	90	95	145
(Treatment water)	30yen/m^3	2,700	2,850	4,350	2,700	2,850	4,350
Others	kg	—	30	—	—	30	—
Silica sand	25yen/kg	—	750	—	—	750	—
Personnel cost	person	0.14	0.14	0.14	0.14	0.14	0.14
	12,300yen /person	1,722	1,722	1,722	1,722	1,722	1,722
Grand total		11,342	20,983	18,956	30,437	20,983	35,036

250yen = 1dollar

4.3 DISTRIBUTION AND DIFFUSION PROMOTION MEASURES

4.3.1. IMPROVEMENT IN THE DISTRIBUTION SYSTEM

All sludge produced in the City that has been composted since 1972 has been restored to green zones and agricultural areas. The factors which have enabled this to be possible are as follows.

a) **Demand Increase**

The amount of organic matter used in agricultural lands in Japan has been drastically reduced due to the tendancy of high dependency on chemical fertilizer and agricultural chemicals and to the difficulty in obtaining organic matter as a result of the lack of labor to produce farm manure. This has resulted in the deterioration of the soil. Under these circumstances, the merits of organic matter has started to draw attention and it has lead to the increase in demand for compost.

b) **Distribution System**

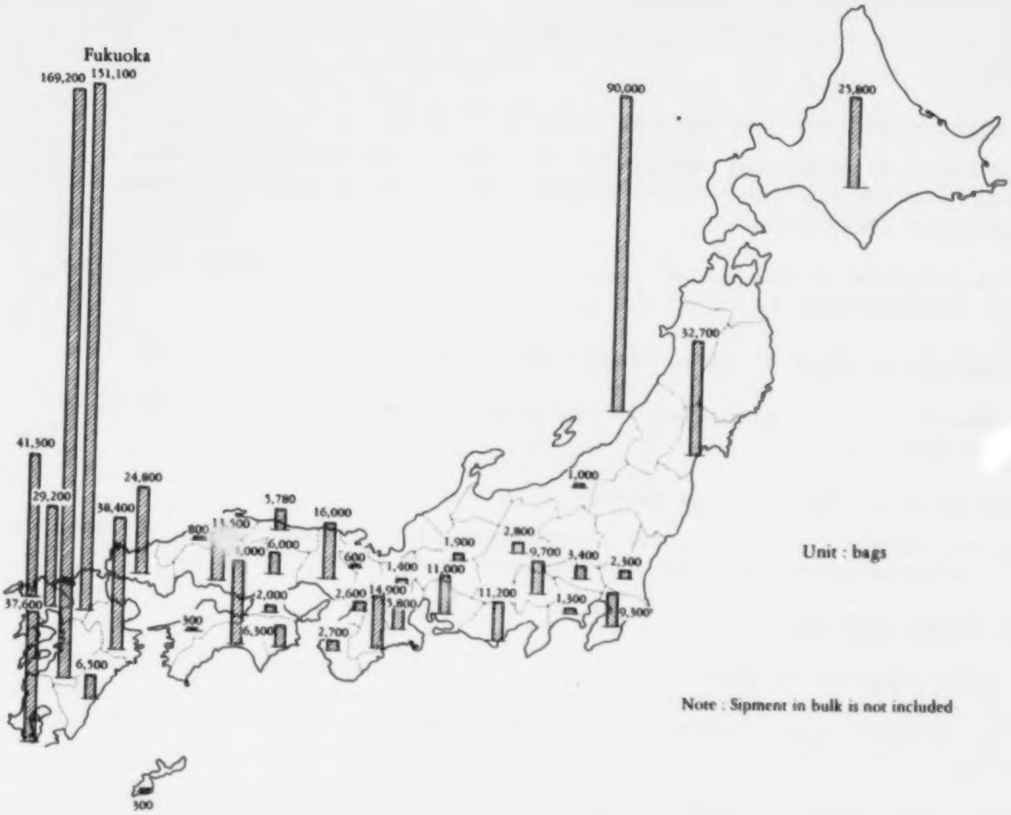
In the case of compost, distribution range is limited due to high transportation cost. Improvement in the distribution system, however, to sell compost in large quantities by private corporations has contributed greatly to the promotion of compost diffusion. Figure 17 shows the distribution system which is similar to the case of chemical fertilizers, i.e., production-storage-sales-use. The shipped amount of compost in 1984 is shown in Figure 18 which indicated that consumption of compost is nationwide.

Treatment plants (Average moisture content 65%) → Sludge Control Center (Primary fermentation, Moisture content 40%) → Private consignment (Secondary fermentation, Moisture content 30%) → Pulverization → Sorting → Bagging → Warehouse storage → Shipment → Wholesaler → Primary store → Retailer → Consumer

Figure 17. Distribution system.

Uses of compost are: vegetables (30%), fruits (20%), golf courses (15%), pasturage (10%), paddy rice (6%), public green zones (4%) and others (10%).

The final consumer price differs according to transportation distances. In the case of 20kg-bagged compost transported within 100km (inside Fukuoka Prefecture), it is 500 yen. The price increases as the distance increases.


Fukuoka
169,200
151,100
90,000
25,800
32,700
41,300
29,200
37,600
38,400
24,800
6,500
800
13,500
1,000
5,780
6,000
16,000
2,000
6,300
300
600
1,400
2,600
14,900
3,800
2,700
1,900
11,000
1,000
2,800
9,700
3,400
2,300
11,200
1,300
9,300
300

Unit : bags

Note : Sipment in bulk is not included

Figure 18.Shipment distribution of product compost. (May 1984~March 1985)

4.3.2 DIFFUSION PROMOTION MEASURES

It has been only some ten years since the compost produced in the City was first put on the market. It is thought to be needed not only to improve the quality of compost but also to stabilize the use of it for the continued composting of sludge. To promote diffusion of compost, measures such as the follows are being implemented.

a) Compost Variety

Products of as many as fourteen varieties in particle size and moisture content according to their use such as paddy rice, vegetables, golf cources and hortculture or to sprinkling methods such as machine sprinkling and hand sprinkling are produced to meet each demand specifically. It is considered, however, to be necessary to further expand the variety in the future to meet demands in better ways. Sales in bulk is also conducted in suburban areas.

b) Public Relations and Education

For the promotion of the use of compost, efforts as follows are being made.

(1) Lectures on needs for compost and its effects

Lectures on usage and management of compost and its constituents or consultation meetings on heavy-metal issues are conducted. Usage leaflets are distributed.

(2) Presentation of the users' experiences to learn their views and ideas and agricultural fairs of the users are held.

(3) Exhibition lands by agricultural-education organizations are set up.

(4) Compost for home gardening is distributed free at events on sewage and agriculture.

c) Use in Urban Public Green Zones

As an organic fertilizer, the compost produced in the facility is used in all afforestation projects of parks and roads conducted by the City.

4.3.3 USAGE GUIDANCE

Usage guidance is given to the users of the compost with the cooperation of agricultural-education organizations and distributors of the compost.
The compost usage research conducted by the City has proved the effects as follows.

(1) It brings about increase in activated soil microorganism and restrain soil pathogenic bacteria.

(2) It helps increase yield of green vegetables which require high ratio of nitrogen.

(3) It improves qualities of fruits and fruit vegetables by enhancing sweetness in them.

(4) It darkens the color (green) of lawns in proportion to the amount used.

(5) Its effect is enhanced by being used with potassic fertilizer.

(6) Lime contained in the compost prevents acidification of soil, increases the gaseous phase, forms aggregated structure, improves physical and chemical properties of soil.

The compost, however, raises the pH of soil when used in great amount and it requires appropriate amounts to be used as shown in Table 15.
It is recommended not to use the compost, though, for soil of over 6.5 pH since it is necessary to sprinkle and mix the compost thoroughly with the soil before planting or sowing. (See TABLE 15)

TABLE 15. Standard amount of compost to be used

Farm products	Yearly amount used per 10 are	Use
General products	400kg	Do not use for soil of over 6.5 pH Mix thoroughly with soil.
Rice, Wheat, Soybean	250kg	〃
Fruit trees	300kg	〃
Tea trees	250kg	〃
Mulberry trees	600kg	〃
Potted plants	10% of soil or 300kg	〃
Lawn	300kg	
Newly cultivated field	2000kg	This amount should be wed only for three years. After initial three years, follow the above amount for use.

4.4 REGULATIONS CONCERNING HEAVY METAL CONTENT

In Japan there are two kinds of regulations concerning heavy metal: the heavy metal concentration regulations in compost prescribed by the Fertilizer Control Act and the regulations controlling total quantity of heavy metals in soil based upon the Soil Control Act.

4.4.1 HEAVY METAL CONCENTRATION IN COMPOST

The Fertilizer Control Act classifies fertilizer made from sewage sludge as special fertilizer and it is subject to the heavy metal concentration regulations. (See TABLE 16)

TABLE 16. Heavy metal content in compost

		Compost in Fukuoka City (1984 year)			Standards
		Average	Maximum	Minimum	
mg per kilogram of dried compost	As	1.3	2.9	0.5	Not to exceed 50
	Hg	0.69	0.91	0.4	〃 2
	Cd	1.5	2.6	0.94	〃 5
	Pb	26	47	19	
	Cr	20	30	16	
	Cu	170	200	160	
	Zn	720	990	390	
	Ni	22	10	28	
Solution extraction test mg per liter of sample liquid	Arsenic or its compounds	ND	ND	ND	Not to exceed 1.5
	Cyanides	ND	ND	ND	〃 1.0
	Mercury or its compounds	0.0014	0.0023	0.0008	〃 0.005
	Cadmium or its compounds	ND	ND	ND	〃 0.3
	Lead or its compounds	ND	ND	ND	〃 3.0
	Sexivalent chromium compounds	ND	ND	ND	〃 1.5
	PCB	ND	ND	ND	〃 0.03
	Organic phosphorous compounds	ND	ND	ND	〃 1.0
	Mercury alkylides	ND	ND	ND	Should not be detected

ND : Not detected

The regulations provide that the content of As(arsenic), Hg(mercury), and Cd(cadmium) in dried fertilizer should not exceed 50 mg/kilogram, 2 mg/kilogram, and 5 mg/kilogram respectively; furthermore the solution extraction regulations protect soil from the solution extraction of heavy metals.

All compost produced in Fukuoka City is tested twice a month to ensure safety. From this test, positive results were obtained as Figure 19 suggests: heavy metal contents were constantly lower than the limit required by the regulations and it remained stable throughout the year. (See Figure 19)

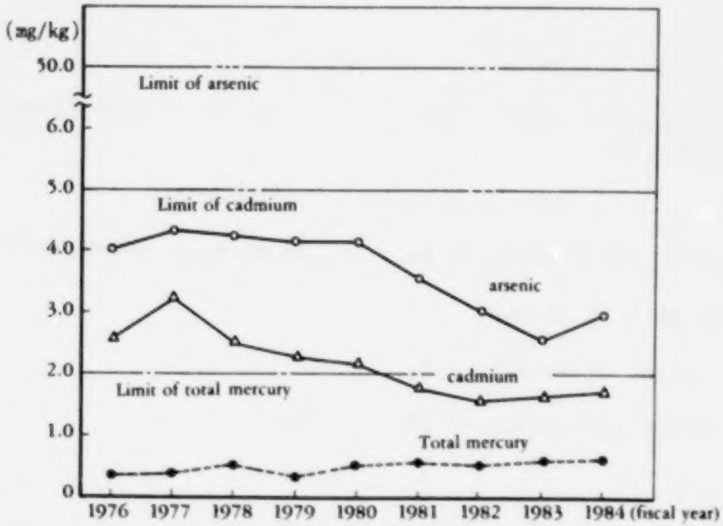


Figure 19.Change of heavy metal content in compost.

4.4.2 REGULATIONS CONCERNING HEAVY METAL CONTENT IN SOIL

The permissible limit level of Zn(zinc) in soil was established in 1984: the content of Zn in dried soil should not exceed 120 mg/kg·DS. This regulation ensures the safety of compost, and thus prevents soil contamination caused by the accumulation of Zn.

TABLE 17. Guide lines for level of Zn, Cu, and Ni

(Unit : mg/dried soil kg)

	Permissible limit level in soil		Maximum permissible level to soil (See the note)			
	Japan	West Germany	U. S. Department of Agriculture	Canada	U. K.	Holland
Zn	120	300	223	162	560	81
Cu		100	112	75	280	20
Ni		50	112	16	128	20

Note : Calculated based upon the guide lines regarding the permissible limit level.

The following formula shows how many times it is possible to harvest out of 400 kilograms of compost.

Assuming that

1) The content of Zn in compost is 720 mg/kg·DS.

2) The amount of compost used in soil is 400 kg per 10 ares (compost contains 30% of moisture.)

3) The content of Zn in soil is 60 mg/kg of dried soil.

4) The permissible limit of Zn in soil is 120 mg/kg of dried soil.

5) The depth of soil is 15 cm.

6) The volume weight of soil is 0.9.

7) The accumulation percentage of Zn is 100%.

Thereby

$$\frac{X(\text{harvests}) \times 0.4(\text{tons/harvest}) \times (1-0.3) \times 720(\text{g/ton})}{0.15\text{m} \times 1{,}000\text{m}^2 \times 0.9} = 120 - 60$$

$$X=40$$

This formula suggests that it is possible to harvest at least 40 times out of 400 kg of compost per 10 ares.

However, it could be possible to harvest more than that because some amount of Zn can drain out or might be absorbed by crops.

There have been no reports claiming that soil contamination was caused by compost.

The behavior of heavy metals in soil depend upon a number of factors: the amount of compost, the pH value of compost, the content of organic substance, soil property, and the category of crops. These complex factors make it necessary to conduct further detailed surveys.

In view of the fact that compost produced in the City is shipped to almost every part of Japan, Fukuoka is planning to establish the nationwide network to research the soil condition in farmland where compost is used.

4.4.3 MEASURES TO REDUCE HEAVY METAL CONTENT IN SLUDGE

Simply satisfying the regulations will not protect farmland from soil pollution; there is no doubt that the less heavy metal content in compost is the better both for safety of soil and for sales promotion of compost. In this respect the City set up the following measures to assure the safety of compost and to promote the recycling of sludge:

a) Control over Facilities Discharging Waste Water Containing Heavy Metals

The most effective way to reduce heavy metals in sewage is to check the discharge of heavy metals into the sewers at their resources.
In Fukuoka City, facilities letting out large amount of waste water are designated as Special Facilities. There are about 670 such facilities. Out of these Special Facilities, the facilities which are under the application of the standards described in Table 18 are required to submit a report on their water analysis and maintenance data. To further enhance the control of heavy metal discharge, the City conducts water sampling inspections at these facilities. Moreover, at such facilities as research institutions, hospitals, and metal plating plants, which discharge large amount of heavy metals, the City carry out on-the-spot inspections and water sampling inspections; beside that, these facilities are required to use professional waste water and sludge collectors.

TABLE 18. Effluent standards into sewerage

Chemical substance	Unit	Specially designated facilities		Other facilities	
		Amount of droinage discharge			
		$50m^3$ or more per day	Less than $50m^3$ per day	$50m^3$ or more per day	Less than $50m^3$ per day
Cadmium or its compounds	mg/ℓ	0.1	0.1	0.1	0.1
Cyanides	〃	1	1	1	1
Organic phosphorous compounds	〃	1	1	1	1
Lead or its compounds	〃	1	1	1	1
Sexi-valent chromium compounds	〃	0.5	0.5	0.5	0.5
Arsenic or its compounds	〃	0.5	0.5	0.5	0.5
Mercury or its compounds	〃	0.005	0.005	0.005	0.005
Mercury alkylides	〃	Not detectable	Not detectable	Not detectable	Not detectable
PCB	〃	0.003	0.003	0.003	0.003

b) Prevention of Heavy Metal Mixture in Dehydration Process

Lime and ferric chloride are applied as dehydration auxiliaries in the dehydration process. Fukuoka has the standards for these two auxiliaries in compost to assure safety; only the auxiliaries that passed the standard tests conducted by such a public agency as the Pollution Analysis Center can be used in the dehydration process.

TABLE 19. Content standards of heavy metal in Dehydration Auxiliaries

Lime

	Content standards	Actual content
Cd	Not to exceed 2.5mg/kg	1.88 (2.3～0.87)
As	〃 2.5mg/kg	0.92 (1.3～0.5)
T-Hg	〃 0.04mg/kg	Less than 0.01

Figures in the parenthesis indicate the range.

Ferric chloride

	Content standards	Actual content
Cd	Not to exceed 0.3mg/kg	0.2 (less than 0.3～0.03)
As	〃 15mg/kg	0.6 (less than 1.3～0.5)
T-Hg	〃 0.005mg/kg	0.002 (less than 0.05～0.001)
Pb	〃 3.0mg/kg	0.9 (less than 2.0～0.3)
Cu	〃 30mg/kg	1.5 (less than 2～1)
Zn	〃 50mg/kg	29.7 (44～18)
T-Cr	〃 20mg/kg	9.0 (13～2)

c) Others

Fukuoka City is working on the development of heavy metal separation methods in cooperation with the Hydolic and Sanitary Engineering Research Laboratory (headed by Dr. Hanajima) at Fukuoka University.
A new kind of heavy metal separation technique using floatation machines is under experiments. Promising results have been obtained so far. These results prove that it is possible to separate considerable amount of heavy metals from used water. Further detailed researches are being carried on aiming at the practical adaption of this technique.

4.5 RESEARCHES INTO THE EFFECTS OF COMPOST ON SOIL AND CROPS

4.5.1 THE RESULTS OF RESEARCHES

Since 1977 the City has been conducting extensive researches into effects of compost used in fields and paddies in cooperation with agricultural research organizations and university research laboratories.

The main research subjects are as follows:

(1) The effects on the yield and soil improvement

(2) The examination of adequate applying technique for compost and of applying limit of compost

(3) The accumulation of heavy metals in soil

(4) The absorption of heavy metals by crops

(5) The effects on growth of crops

The followings are the reports on the effects of compost used in paddy fields and newly developed agricultural fields.

a) Tests in Paddy Fields

(1) Research Method

The compost section and the control section were set up in 1977: in the former section 240 kilograms of compost is used per 10 ares and in 18.3 ares of the latter section only chemical fertilizer is used. Since then, the long-term comparative study has been made in these two sections.

(2) Results

As TABLE 20 indicates, the stalk diameter of rice in the compost section was 6% bigger than that in the control section in 1984; the subterranean portion in the compost section grew by 68%, which means that the fall-resistancy of rice in this section improved a great deal.
The yield in the compost section has become stable recently, and has exceeded that in the control section by 10%, which indicates that the use of compost is effective.

TABLE 20. Study of growth condition of paddy rice 1980-1984

	Weight of terrestrial portion	Weight of sudterra-nean portion	Number of rice seeds per rice stubble	Total weight of rice seeds per rice stubble	Length of head	Length of stalk	Number of heads per rice stubble	Total weight of 1,000 rice seeds	Number of seeds per head	Diameter of stalk	Unhulled rice /straw
1980	95	86	104	104	101	97	99	100	—	—	112
1981	88	90	93	98	98	99	97	103	—	—	108
1982	104	108	109	110	101	104	114	101	108	102	105
1983	104	120	104	104	101	101	97	100	108	108	104
1984	105	168	114	109	99	107	106	96	108	106	104

Note : The base figure of 100 is the figure of each item in the control section

(3) Behavior of Heavy Metals

Among heavy metals, Zn(sinc) can be absorbed most easily by plants. In the compost section, the amount of Zn in the edible portion of rice increased by 20% in these eight years. However, such harmful heavy metals as As(arsenic), Hg(mercury), and Cd(cadmium) were absorbed only by the subterranean portion; these heavy metals were not detected in the edible portion.
Though it is eight years since this test started, no distinct difference in heavy metal content was found between in the compost section and in the control section. One reason for this could be that the amount of compost was limited to 240 kilograms per 10 ares (which is well within the adequate limit for paddy fields.) For another reason, the water in paddies may have some influence on the behavior of heavy metals.

TABLE 21. Heavy metal content in crops

(mg/kg : dried)

		Polished rice	Brown rice	Unhulled rice	Leaf and stalk	Subterranean portion
As	Control section	Not detected	〃	〃	〃	12
	Compost section	Not detected	〃	〃	〃	9.8
Hg	Control section	Not detected	〃	〃	〃	〃
	Compost section	Not detected	〃	〃	〃	〃
Cd	Control section	Not detected	〃	〃	〃	0.61
	Compost section	Not detected	〃	〃	〃	0.23
Cr	Control section	Not detected	〃	〃	1.7	49
	Compost section	Not detected	〃	〃	1.6	40
Pb	Control section	Not detected	〃	〃	〃	10
	Compost section	Not detected	〃	〃	〃	10
Cu	Control section	3.9	4.1	3.9	6.7	69
	Compost section	4.2	4.5	4.0	6.8	55
Mn	Control section	21	26	58	500	340
	Compost section	20	27	55	440	440
Fe	Control section	17	23	42	240	57,000
	Compost section	17	24	41	330	66,000
Zn	Control section	25	24	23	20	82
	Compost section	25	27	19	20	84
Ni	Control section	Not detected	〃	〃	〃	26
	Compost section	Not detected	〃	〃	〃	26

Note : ten point average

(4) Another Test

A large amount of compost was given to soil in bench scale test every year and the effects were studied. From this test, the following results were obtained:

1) The productivity of soil improved.

2) The alkalinization in soil advanced.

3) The growth of rice was restrained at early stage, but it recovered later.

4) Heavy metal density in crops didn't increase.

TABLE 22. Analysis of brown rice grown and soil after harvest

Item / Nature of soil		Soil after harvest				Brown rice					
		pH	T-N	T-C	Base saturation percentage	N	P	K	Cd	Zn	Cu
	kg/10 are		%	%	%	%	%	%	mg/kg	mg/kg	mg/kg
Clay loam	0	5.3	0.19	1.64	84	1.38	0.37	1.11	<0.01	19.10	2.84
	400	6.6	0.19	1.58	148	1.34	0.35	1.01	0.03	18.75	2.97
	800	7.0	0.18	1.62	177	1.37	0.35	1.03	0.02	17.95	2.78
	1,600	7.6	0.20	1.73	253	1.33	0.35	1.08	<0.01	23.90	4.07
Loamy Sand	0	5.9	0.12	0.96	63	1.25	0.37	1.08	0.03	18.76	2.94
	400	6.7	0.14	1.12	109	1.29	0.33	1.03	0.02	18.38	2.67
	800	7.3	0.15	1.30	155	1.26	0.36	1.00	0.04	19.07	3.47
	1,600	7.9	0.13	1.55	330	1.31	0.36	1.01	0.02	18.32	3.03

b) Tests in Newly Developed Fields

(1) Research Method

Newly developed fields were divided into three sections: 1.8 ares of the control section in which chemical fertilizer was applied, the compost section in which 2tons of compost were used per 10 ares for the first and the second year (lime in compost was reduced and calcium carbonate was added instead), and the farm manure section. In each section, taros were planted in the first year, turnips in the second year, and potatoes in the third year.

(2) Results

Crops in the compost section grew more thrivingly than those in the control section. The yield in the compost section was 15% better than that in the control section. Though the total yield of potatoes in the compost section was less than that in the farm manure section, the yield of large- and medium-sized potatoes surpassed that in the farm manure section.

The quality of potatoes is measured by the starch value. Potatoes grown in the compost section showed the highest starch value. This suggests that compost has effects on improvement of the quality of potatoes.
In the compost section, nitrogen content in crops increased; however phosphorus and potassium content was smaller than that in the control section. (See TABLE 23, 24)
The pH value and the base saturation percentage in soil of the compost section went up slightly by the effect of lime contained in compost. The total nitrogen and carbon rose in the first and the second year. In the third year, the total nitrogen went down to the same level of that in the control section.
In soil of the compost section, the true density became smaller, the air phase became amplified, and the porosity and the coefficient of permeability went up. These results indicate that compost upgraded the soil condition.

(3) Behavior of Heavy Metal

In the compost section, heavy metal content in stems and leaves rose slightly, but the edible portion had less heavy metals than that in the control section.
The heavy metal content in soil of the compost section did not show any increase after three years. There was little difference between heavy metal content in compost section and that in the other two sections. (See TABLE 24, 25)

TABLE 23. Growth condition and yield of potatoes

	Growth				Yield						
	May 8		June 18								
Item	Length of stem	Number of stems	Length of stem	Number of stems	Weight of stem (dried)	Large-sized potato (120gr and more)	Medium-sized potato (60-120gr)	Small-sized potato (less than 60gr)	Total weight of potatoes	Total weight of dried potatoes	Starch value (%)
	cm		cm		kg/are	kg/are	kg/are	kg/are	kg/are	kg/are	
Control section	10.0	1.90	61.2	1.85	19.34	346.3	118.0	60.8	525.1	93.7	11.7
Farm manure section	12.0	1.95	64.7	2.30	16.28	371.7	168.3	106.9	646.8	109.7	10.9
Compost section	11.1	1.75	55.4	2.10	20.25	364.0	180.0	63.3	607.3	113.7	13.1

TABLE 24. Analysis of potatoes

(% : dried)

		Water (%)	N	P	K	Ca	Mg	Heavy metals (mg/kg)		
								Cd	Zn	Cu
Stem, Leaf	Control section	89.7	3.41	0.37	4.83	2.06	0.89	1.39	45.8	8.28
	Farm manure section	90.8	3.77	0.38	4.83	1.32	0.92	1.76	59.8	14.67
	Compost section	89.5	3.48	0.33	4.14	2.28	1.09	1.38	49.6	8.91
Potato	Control section	82.2	1.25	0.19	1.65	0.04	0.07	0.19	31.8	3.37
	Farm manure section	83.0	1.28	0.21	1.65	0.03	0.07	0.14	22.2	4.07
	Compost section	81.3	1.45	0.20	1.57	0.04	0.07	0.17	18.7	3.18

TABLE 25. Analysis of soil after harvest

	pH	T-C	T-N	Ex-Ca	Perchloric acid decomposition			True density	Three phases			Available water	Coefficient of water permeability
					Cd	Zn	Cu		Solid phase	Liquid phase	Air phase		
		%	%	me/100g	mg/kg	mg/kg	mg/kg		%	%	%		
Control section	7.7	0.38	0.06	11.4	0.12	100.8	30.2	2.64	51.6	17.5	30.9	12.1	1.1×10^{-3}
Farm manure section	7.0	0.35	0.04	7.3	0.11	108.5	32.4	2.60	48.8	16.6	34.6	13.9	1.5×10^{-3}
Compost section	7.9	0.32	0.04	13.1	0.17	113.1	33.8	2.62	43.1	15.9	41.0	12.2	3.2×10^{-3}

4.5.2 FUTURE RESEARCHES

Researches carried out so far proved that compost improves soil condition and productivity. It is being planned to conduct the following researches regarding heavy metal behavior.

(1) Researches to find out the appropriate amount of compost to be used in farmland, how the accumulation rate of heavy metals varies by the nature of soil, and how the accumulation rate in crops changes by the category of crops

(2) Researches concerning activity of heavy metals and absorption of heavy metals by crops in the acid soil. No tests as yet have found heavy metals in the edible portion of crops in the compost section. However it is suspected that this is because the alkalinization of soil might be preventing crops from absorbing heavy metals. Consequently it is necessary to observe heavy metal behavior in soil when soil turns into acid soil.

(3) Researches into multiplex effects of compost

5 SUMMARY

As stated in 1 INTRODUCTION, Fukuoka City believes that turning sludge into compost is the most adequate sewage disposal method because

1) sludge contains useful organic and inorganic substances;

2) sludge in the city is safe to use as compost;

3) there still remains farmland in many parts of the city;

4) this method is appropriate from a long-term viewpoint.

The City has made efforts to improve compost quality and to expand distribution network with the cooperation of SANKYO YUKI CO., LTD. to which the City entrusted treatment of sludge. Fukuoka City has been also trying to reduce heavy metal content in sewage.

These efforts have now come to bear fruit: 100% of sludge in the City has been made into compost since 1972, and compost produced in the City is shipped to various parts of Japan.

Fukuoka City holds that Japan, being scarce in natural resources, should make use of sludge to improve soil productivity.

Varied researches have been conducted past eight years with the purpose of estimating the effects of compost and grasping heavy metal activities in soil. The City is projecting to carry out further detailed researches working together with such administration agencies as the Ministry of Construction and the Ministry of Agriculture, Forestry and Fishery, as well as with research institutions at universities.

Tenth United States/Japan Conference
on Sewage Treatment Technology

TECHNICAL EVALUATION OF "ENERGY-SAVING" AERATION DEVICES

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1. PRESENT SITUATION OF AERATION DEVICES IN JAPAN

As of the end of FY 1982, there are 529 publicly owned wastewater treatment plants in service in Japan. Table 1 shows their classification by treatment processes.

Table 1 Classification of wastewater treatment plants by processes

(FY 1982)

Process	Public sewerage	Basin-wide sewerage	Designated public sewerage	Designated environ-mental sewerage	Total
Conventional activated sludge	331	35	3	1	370
Step aeration	70	6	1	-	77
Extended aeration	10	-	-	3	13
Contact stabilization	1	-	-	-	1
Oxygen aeration	5	-	-	-	5
Rotating biological contactor	4	-	1	7	12
Oxidation ditch	5	-	-	2	7
High-rate trickling filter	13	-	-	-	13
High-rate activated sludge	19	-	-	-	19
Chemical coagulation	2	-	-	-	2
Plain sedimentation	7	-	3	-	10
Total	467	41	8	13	529

As shown, 491 plants, or 93% of all, adopt activated sludge process, including modified processes.

Sewerage construction in Japan has so far been pushed forward in densely populated areas through the construction of comparatively large plants, and the activated sludge process has taken a leading role to cope with a limited availability of land.

Of the 491 wastewater treatment plants with activated sludge process, those using the mechanical aeration devices number only 46, and the remaining 445 are with diffused aeration devices.

The specifications of the diffused aeration devices employed in Japan are described in the "Guidelines for Sewers and Wastewater Treatment Plants,[1)]" as summarized in Table 2.

Most of diffused aeration devices used in Japan are plate or pipe diffusers capable of generating fine bubbles. The maximum pore size of the plate diffuser is about $400\mu^{\circ}$, and the standard air flow rate is 80 to 100 ℓ/min per plate. The larger air flow rate increases the bubble sizes, reducing the oxygen transfer efficiency. This in turn is likely to increase air flow resistance due to clogging. The maximum pore size of the pipe diffuser is about $450\mu^{\circ}$, and the standard air flow rate is 120 to 150 ℓ/min per pipe. In most cases, these diffusers are installed on a sidewall along the bulk flow direction for a spiral flow configuration in the aeration tank.

Table 2 Specifications for diffusers

Type / Material / Item	Fine bubble					Coarse bubble		
Material	Porous ceramics			Unplasticized plastics		Stainless steel (SUS304)	Cast iron, ABS resin	Ductile cast iron and PVC
Item	Plate diffuser	Pipe diffuser	Disk diffuser	Plate diffuser	Pipe diffuser	Perforated pipe	Sparger	Disk diffuser
Standard air flow rate (ℓ/min)	80 to 100/sheet	120 to 150/ piece	30 to 40/ piece	80 to 120/sheet	100 to 150/ piece	300/piece	250/ piece	200/ piece
Pore size (μm)	Max. 400	Max. 450	avg. 150	200 to 500	200 to 500	dia. 4.0 mm	6.3 mm	Max. gap, 4.0 mm
Dimensions (mm)	⊡ 300 x 300 x 30 t	75 o.d. x 50 i.d. x 500 L	178 o.d. x 38 H x 19 t	⊡ 300 x 300 x 30 t	75 o.d. x 50 i.d. x 500 L	25 o.d. x 650 L x 15 holes	-	100 o.d.
Weight (kg/unit)	4	2	1.3	1.8	0.8	1.7	0.2 (ABS)	0.6

Those fine-bubble aeration devices have been in use since 1928. But there is little information available about service life of plate diffusers concerning replacement frequency. Figure 1 shows annual changes in air flow resistance of the plate diffusers measured by the Public Works Research Institute, Ministry of Construction, for 14 years at the Ochiai Wastewater Treatment Plant, Tokyo.

While abrupt changes in air flow resistance were noticed during a total period of 10 intervening years, they would have been not so serious a problem.

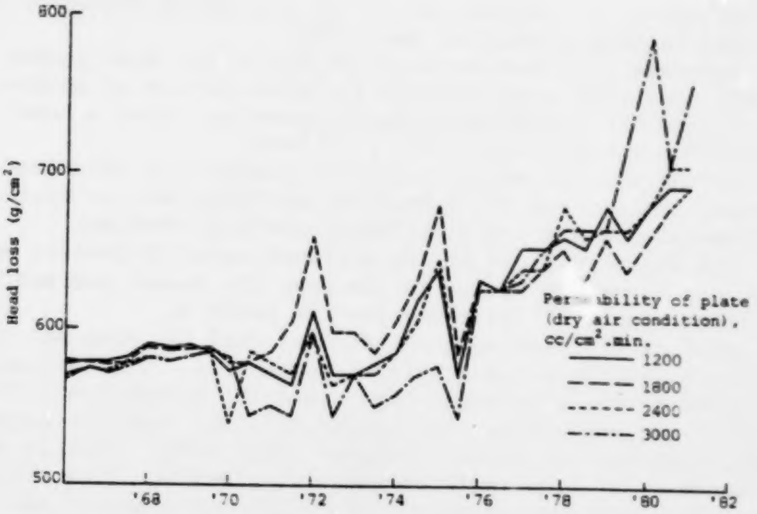


Fig. 1 Yearly changes in head loss of four different plate diffusers

The clogging degree of plate diffusers is dependent on the characteristics of influent wastewater and the operating conditions of aeration tank (blower shutdown frequency, concentration of dissolved oxygen (DO), air flow rate etc.), and their service life varies from plant to plant. As the wastewater treatment plants constructed in the past ten years account for more than 50% of all the plants, the renewal of diffused aeration devices will grow in future.

2. BACKGROUND TO DEVELOP ENERGY-EFFICIENT AERATION DEVICES

The total electric energy generated in Japan in FY 1982 was 581 x 10^9 kWh, of which 2.9 x 10^9 kWh, or 0.5%, was consumed by sewerage works. At present, public sewer-covered population in Japan is as low as 33% of the total. With increase in the sewer coverage, the power consumption by sewerage works will increase.

Table 3 shows the breakdown of operation and maintenance costs for sewerage works all over Japan in FY 1982. As shown, the electric bills are the largest of all the cost items, registering 24.1%. This indicates that the reduction of power consumption has a significant effect on the reduction of overall expenditure.

Table 4 is a summary of the survey of the power consumption of at six wastewater treatment plants which employ either conventional activated sludge process or step aeration process, both being typical processes in Japan. At the plants sludge treatment processes consist of gravity thickening, anaerobic digestion, elutriation, and mechanical dewatering. As shown in Table 4, power to operate aeration blowers are found to account for an average 45% of total plant power consumption, suggesting that the improvement of energy efficiency of aeration devices will be directly effective for the reduction of plant operating costs.

Table 3 Breakdown of operation and maintenance costs of sewerage works (FY 1982)

(in thousand yen)
(%)

Item \ Facility	Sewer	Pump station	Treatment plant	Total
Electricity	-	10,046,348	49,099,672	59,146,020
	-	(28.9)	(26.4)	(24.1)
Labor	-	-	40,009,193	40,009,193
	-	-	(21.5)	(16.3)
Commission	7,374,389	4,142,551	34,259,157	45,776,097
	(29.6)	(11.9)	(18.4)	(18.6)
Cleaning	2,019,404	-	-	2,019,404
	(8.1)	-	-	(0.8)
Repairs	6,682,654	3,687,709	15,138,212	25,508,575
	(26.8)	(10.6)	(8.1)	(10.4)
Chemicals	-	-	13,925,793	13,925,793
	-	-	(7.5)	(5.7)
Fuel	-	276,937	8,998,550	9,275,487
	-	(0.8)	(4.8)	(3.8)
Others	-	-	10,874,455	10,874,455
	-	-	(5.9)	(4.4)
Subtotal	16,076,447	18,153,545	172,305,032	206,535,024
	(64.5)	(52.2)	(92.7)	(84.1)
Administration	8,857,531	16,607,756	13,559,717	39,025,004
	(35.5)	(47.8)	(7.3)	(15.9)
Total	24,933,978	34,761,301	185,864,749	245,560,028
	(100.0)	(100.0)	(100.0)	(100.0)

Table 4 Breakdown of power consumption

(%)

Treatment plant	Pumping	Wastewater treatment	Sludge treatment	Others	Blower
1	-	-	17.2	-	44.8
2	21.9	-	-	-	43.3
3	22.5	55.5	9.9	12.1	40.3
4	16.7	70.2	10.4	2.7	61.3
5	25.1	60.4	12.9	1.6	-
6	24.8	53.9	17.2	4.0	34.7
Average	22.2	60.0	13.5	5.1	44.9

3. AN INTRODUCTION TO THE AERATION DEVICE EVALUATION SYSTEM

3.1 Construction Technology Assessment System

In 1978, the Ministry of Construction introduced the Construction Technology Assessment System for implementation of advanced technologies for public works. The underlying purpose of this system is to assess new technologies and to reflect the findings in the public works. The details of the system and the technologies evaluated under the system by FY 1983 were already reported by Nakamoto at the 9th U.S.-Japan Conference on Sewage Treatment Technology.

On agenda for FY 1984 and FY 1985 in relation to sewage works are the following projects.

FY 1984: "Development of solid-liquid separation devices by screening method"

FY 1985: "Development of dissolved oxygen sensors for automation of aerator control"

The evaluation of energy-efficient aeration devices was conducted in FY 1982 under this evaluation system.

3.2 Goals of Development

The development of energy-efficient aeration devices was promoted to achieve the following four major goals.

(1) Reduction of power consumption by 20% as compared with the "conventional" diffusers (plate and pipe diffusers).

(2) Reduction of biochemical oxygen demand (BOD_5) and suspended solids (SS) in the effluent to less than 20 mg/ℓ and 70 mg/ℓ, respectively, when the diffusers are installed in the activated sludge reactor for the secondary treatment of domestic wastewater.

(3) Easy adaptation to the existing aeration tanks.

(4) Improved operation, and easier inspection, repair and parts replacement.

3.3 Evaluation Procedure

The evaluation was made based on the test results submitted by the applicants, and the Construction Minister accorded a certificate of approval to the accepted products.

Prior to the invitation of manufacturers to the assessment, a technology evaluation committee consisting of the engineers and researchers representing the Ministry of Construction and local municipalities and academic authorities from the universities formulated guidelines for the evaluation procedures of testing and data analysis. The guidelines for the testing methods are summarized below.

(1) To demonstrate performances of the new devices by treating actual wastewater either at the actual facility or pilot plant of a comparable scale, by making a comparison between these and conventional plate or pipe diffusers.

(2) To conduct above tests for at least one month each including both summer and winter time operation.

(3) To collect data, including power consumption by aeration device, wastewater flow, oxygen transfer efficiency, distribution of DO and MLSS concentrations in the aeration tank, characteristics of influent, mixed liquor and effluent.

4. RESULTS OF EVALUATION

4.1 Evaluated Devices

The evaluation was made on five types of "new" diffusers. Fig. 2 illustrates the operating principles, structural designs and installation methods of these devices.

The evaluated devices can be classified into two groups as to operating principles.

"Bayer Injector" of Mitsui Shipbuilding & Engineering, "Hanshin Aquarator" of Hanshin Power Machinery, and "Jet Aerator" of Nishihara Environmental Sanitation Research Corporation, are of a type designed to inject air into turbulent wastewater for fining the bubbles, and the resultant high velocity jet of water is used to mix wastewater in the tank. Bayer Injector and Jet Aerator also use a pump to circulate mixed liquor; the former has a liquid nozzle of 8.2 mm, which is a little smaller than enough, and requires a kind of strainer in its water delivery line. Hanshin Aquarator has a submerged impeller dubbed the diffused aeration rotor, which is driven by a submerged motor to generate a circulating water flow.

Aeration device of NGK Insulators and the dome type diffuser of Mitsui-Miike are for the whole floor distributed aeration, designed to generate ultrafine bubbles. Both types are submerged parts, such as riser pipes, made of synthetic resins, and are fitted with a water drain pipe, in order to prevent pipe scale from blocking up diffusers. In case of 260 m° plate diffusers, the air filter is a combination of wet and dry filter elements as it is in wide use currently. In the dome type diffuser, a special filter is used to prevent airborne dust from clogging up the diffuser pores.

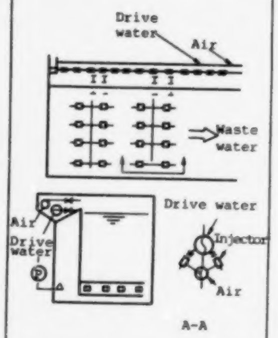
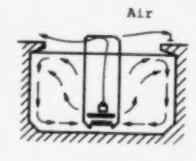
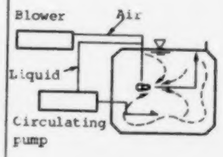
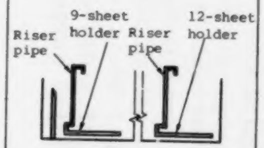
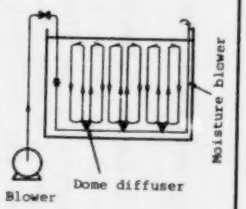
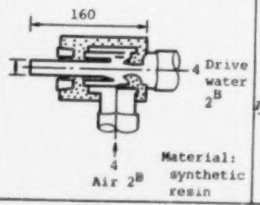
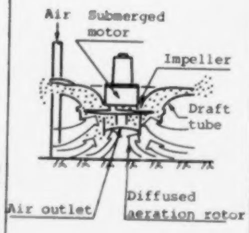
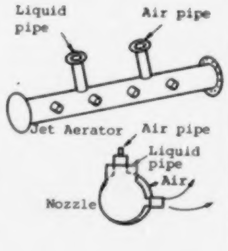
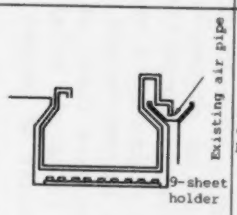
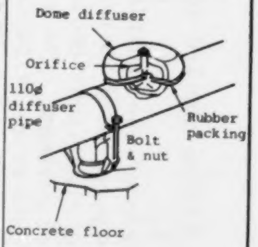
Manufacturer	Mitsui Shipbuilding & Engineering	Hanshin Power Machinery	Nishihara Corporation	NGK Insulators	Mitsui Miike Works
Name of device	Bayer Injector	Hanshin-Aquarator	Jet Aerator	Plate Diffusers	Dome Diffuser
Installation method	 Drive water Air Waste water Air Drive water Drive water Injector Air A-A		 Blower Air Liquid Circulating pump	 Riser pipe 9-sheet holder Riser pipe 12-sheet holder	 Moisture blower Blower Dome diffuser
(Diffuser)	Bayer Injector  160 4 Drive water 2B 4 Air 2B Material: synthetic resin	 Air Submerged motor Impeller Draft tube Air outlet Diffused aeration rotor	 Liquid pipe Air pipe Jet Aerator Air pipe Liquid pipe Air Nozzle	 Existing air pipe 9-sheet holder	 Dome diffuser Orifice 110ø diffuser pipe Rubber packing Bolt & nut Concrete floor
Operating principles	The bubbles are fined by the driving energy of water for high-efficiency oxygen transfer.	Coarse bubbles are mechanically fined down by the diffused aeration rotor, and the impeller performs air-gas mixing as well as agitation.	Jet Aerator delivers a high-velocity jet of the circulating liquid and air for air-liquid mixing and agitation.	Whole floor aeration by plate diffuser (300 x 100 x 10) with 260 µ pores.	Whole floor aeration by dome diffuser with 150 µ pores.

Figure 2. An outline of evaluated devices

4.2 Test Procedure

The testing conditions applied to the newly developed devices are summarized in Table 5.

With the exception of the facility used for Bayer Injector, all the evaluated devices were operated in the aeration tanks in service. Accordingly, the test conditions could not necessarily be standardized for all the evaluated devices. The power consumption was directly measured with a watthourmeter or calculated from air delivery rate, delivery air pressure, etc. to meet specific site conditions.

For the determination of oxygen transfer efficiency, the aeration tank was divided into several compartments, and the respiration rate and DO content in the mixed liquor were measured in each compartment in order to figure out the oxygen demand of the entire aeration tank.

The analytical data were taken at an interval of 2 hours on an around-the-clock basis to obtain total oxygen demand of a day which was divided by the total amount of oxygen supply to determine oxygen transfer efficiency.

Table 5 Test conditions

	Bayer injector		Hanshin aquarator		Jet aerator		Plain plate diffuser		Dome diffuser	
	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility
Wastewater inflow (m^3/day)	781 to 1,350	5,375 to 7,134	14,400 to 21,600	same as tested facility	73,700 to 75,400	77,800 to 80,800	4,000	same as tested facility	6,600 to 7,000	5,800 to 5,900
BOD loading (kg/m^3.day)	0.33 to 0.43	0.23 to 0.32	0.16 to 0.46	same as tested facility	0.37 to 0.68	0.32 to 0.55	0.34	same as tested facility	0.24 to 0.40	0.25 to 0.34
Aeration time (h)	6.7 to 8.9	8.1 to 10.7	5.0 to 7.4	same as tested facility	4.8 to 4.9	5.6 to 6.0	7.0	same as tested facility	6.1 to 6.8	7.5 to 7.6
Submerged depth of aerator (m)	4.5	3.5	4.2		4.6	4.5	3.55	3.75	5.77	5.5
Tank dimensions D: depth W: width L: length	D = 5 W = 5 L = 10	D = 4 W = 5 L = 60	D = 4.5 W = 6 L = 165	same as tested facility	D = 10 W = 10 L = 152	D = 4.5 W = 7.4 L = 152	D = 4 W = 6 L = 52	same as tested facility	D = 6 W = 8 L = 38	same as tested facility
Diffuser*	-	Pipe diffuser	-	Plate diffuser	-	Plate diffuser	-	Plate diffuser	-	Pipe diffuser
Air flow control method	Manual control to keep DO in the center of the aeration tank at 0.5 mg/ℓ during daily peak hours.	Manual control to keep DO at 1 mg/ℓ at the outlet of the aeration tank during 9 to 10 a.m. period.	Manual control so that DO at the outlets of the aeration tanks of both facilities becomes the same.		Manual control to minimize air supply to the extent a required quality level of effluent is maintained. Aeration tank outlet: DO, 1 to 2 mg/ℓ	Aeration tank outlet: DO, 3 to 4 mg/ℓ	Manual control so that the aeration tank outlets of both facilities show the same DO value.		Automatic control to keep DO at 1 or 1.5 mg/ℓ in the center of the aeration tank.	Automatic control to keep DO at 1 or 2 mg/ℓ in the center of aeration tank.
Remarks	The tested facility is a pilot plant installed within the same wastewater treatment plant that is installed with conventional diffusers.		Same plant		Same plant		Same plant		Same plant	

* Same as specified for the ceramic diffusers in Table 2.

4.3 Test Results

(1) Power consumption

The evaluated devices were compared in terms of power consumption per m^3 of effluent or kg of BOD_5 removed. Performance of each device is summarized in Table 6.

Table 6 Summary of power consumption

	Bayer injector		Hanshin aquarator		Jet aerator		Plain plate diffuser		Dome diffuser	
	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility
Power consumption per m^3 of effluent (kWh/m^3)	0.0729	0.0973	0.0732	0.1045	0.0653	0.0994	0.0694	0.1098	0.068	0.098
Power reduction ratio, %	25.1		29.9		34.4		36.8		30.6	
Power consumption per kg of BOD_5 removed (kWh/kg)	0.703	0.949	1.19	1.66	1.002	1.474	0.889	1.422	0.82	1.25
Power reduction ratio, %	26.0		28.3		32.0		37.5		34.4	

The power consumption per m^3 of effluent was 0.065 to 0.073 kWh for the tested devices as against 0.097 to 0.11 kWh for the conventional devices, indicating to achieve 20% power reduction. On the other hand, the power consumption per kg of BOD_5 removed was 0.7 to 1.2 kWh for the tested devices as against 0.95 to 1.66 kWh for the conventional devices, also achieving the 20% power reduction.

(2) Oxygen transfer efficiency

As tested devices were operated by a lower air flow rate than for the conventional devices, it was difficult to make direct comparison between these in terms of oxygen transfer efficiency. Table 7 shows relationship between oxygen transfer efficiency and air supply rate (amount of air supplied into a unit volume of aeration tank per hour).

Table 7 Air supply rate vs. oxygen transfer efficiency

	Bayer injector		Hanshin aquarator		Jet aerator		Plain plate diffuser		Dome diffuser	
	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility	Tested facility	Control facility
Air supply rate ($m^3/m^3.h$)	0.436	0.656	0.229	0.775	0.35	1.11	0.69	1.12	0.51	1.05
Oxygen transfer efficiency (%)	20.5	8.9	27.1	6.7	27.0	10.7	10.2	5.5	23.3	10.4

Air supply rates for the tested devices were 30 to 66% of that for the conventional device. Every new device was proved to have more than twice as high an oxygen transfer efficiency as the conventional ones. However, as they require a power-consuming device other than blower or have a high air flow resistance, the overall power reduction was not as high as the rate expressed by oxygen transfer rate, which was indicated in Table 6 listed earlier.

(3) Mixing performance in the aeration tank

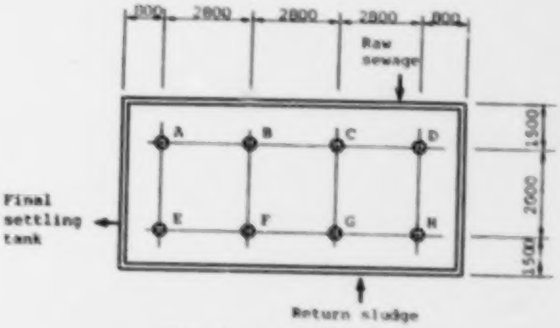
Degree of mixing was studied by the measurement of the distribution of MLSS in the tank. The results of survey are as shown in Figs. 3 through 7. As shown in these figures, every device seems to have a sufficient mixing capability.

Running conditions

Blower capacity: 175 Nm^3/hr

Drive water flow: 39 m^3/hr

Air supply rate: 0.7 m^3/m^3.h



Measuring points

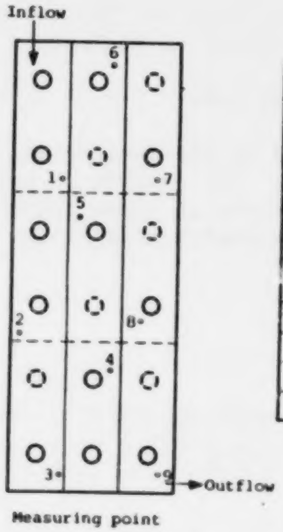
(Unit: mg/ℓ)

Water depth / Measur-ing point	0.5 m	2.5 m	4.5 m
A	1770	1770	1690
B	-	1770	-
C	-	1735	-
D	-	1700	-
E	-	1770	-
F	1800	1870	1800
G	1800	1720	1740
H	-	1735	-

Fig. 3 Distribution of MLSS in the aeration tank during operation of Bayer Injector

Running conditions

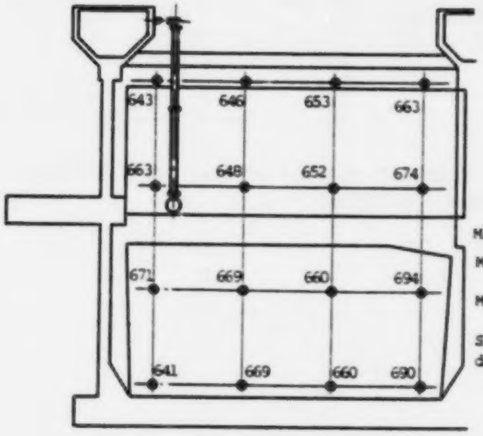
Air supply rate: 0.23 $m^3/m^3.h$



Inflow
6
1
7
5
2
8
4
3
9
Outflow
Measuring point

Measuring point	MLSS	
	Upstream	Downstream
1	1,520 mg/ℓ	1,540 mg/ℓ
2	1,370	1,330
3	1,370	1,340
4	1,240	1,180
5	1,140	1,160
6	1,060	1,090
7	1,090	1,030
8	1,050	1,100
9	1,110	1,100
Average	1,216	1,207

Fig. 4 Distribution of MLSS in the aeration tank during operation of Hanshin Aquarator



643
646
653
663
663
648
652
674
671
669
660
694
641
669
660
690

Running conditions

Air supply rate: 0.39 $m^3/m^3.h$

MLSS

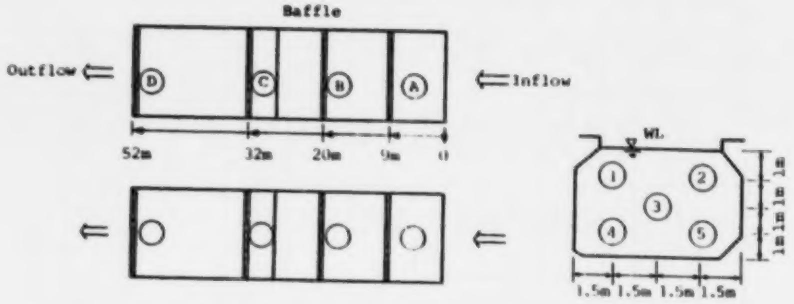
Max. 694 mg/ℓ

Min. 643 mg/ℓ

Standard deviation: 15.4 mg/ℓ

Cross-sectional view of aeration tank

Fig. 5 Distribution of MLSS in the aeration tank during operation of Jet Aerator


Baffle
Outflow
Inflow
D
C
B
A
52m
32m
20m
9m
0
WL
1
2
3
4
5
1m 1m 1m 1m
1.5m 1.5m 1.5m 1.5m

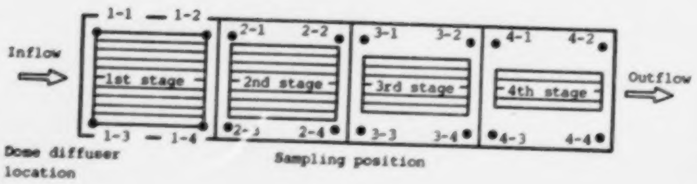
Measuring points in the bulk flow direction

Measuring points on the cross section

Aeration rate: 0.26 $m^3/m^3.h$

	A	B	C	D
1	1570	1620	1710	1640
2	1630	1650	1590	1580
3	1680	1650	1740	1640
4	1580	1660	1660	1540
5	1580	1660	1610	1520

Fig. 6 Distribution of MLSS in the aeration tank during operation of plate diffuser


Inflow
1-1
1-2
1-3
1-4
2-1
2-2
2-3
2-4
3-1
3-2
3-3
3-4
4-1
4-2
4-3
4-4
1st stage
2nd stage
3rd stage
4th stage
Outflow
Dome diffuser location
Sampling position

MLSS distribution measuring conditions

	Air flow rate (Nm³/hr)	Aeration rate (Nm³/hr.m³)
1st stage	138	0.31
2nd stage	110	0.25
3rd stage	88	0.20
4th stage	64	0.15
Total	400	

Sampling position \ Sampling depth		Surface	1 m	2 m	3 m	4 m	Average
1st stage	1-1	1,802	1,948	1,816	1,850	1,848	1,853
	1-2	1,900	1,826	1,900	1,824	1,912	1,872
	1-3	1,914	1,824	1,900	1,860	1,908	1,881
	1-4	1,874	1,880	1,900	1,824	1,876	1,871
	Average	1,873	1,870	1,879	1,840	1,886	1,870
2nd stage	2-1	1,892	1,926	1,904	1,934	1,894	1,910
	2-2	1,854	1,874	1,926	1,926	1,942	1,904
	2-3	1,932	1,886	1,882	1,850	1,928	1,896
	2-4	1,854	1,854	1,876	1,836	1,896	1,863
	Average	1,883	1,885	1,897	1,887	1,915	1,893
3rd stage	3-1	1,812	1,830	1,848	1,876	1,858	1,845
	3-2	2,008	1,920	1,928	2,008	1,938	1,960
	3-3	1,806	1,766	1,806	1,818	1,794	1,798
	3-4	1,876	1,840	1,886	1,934	1,866	1,880
	Average	1,876	1,839	1,867	1,909	1,864	1,871
4th stage	4-1	1,886	1,910	1,924	1,870	1,894	1,897
	4-2	1,928	1,852	1,868	1,906	1,896	1,890
	4-3	1,796	1,888	1,850	1,840	1,880	1,851
	4-4	1,910	1,906	1,926	1,914	1,914	1,914
	Average	1,880	1,889	1,892	1,883	1,896	1,888

Note: Average MLSS in the aeration tank: 1,881 mg/ℓ

Figure 7. Distribution of MLSS in the aeration tank

(4) Effluent quality

There was little difference in the removal efficiency between two facilities one with new diffusers, the other with conventional devices as far as effluent quality was concerned. No significant difference was also noticed in the settleability of activated sludge between them.

4.4 Conclusion

From the results of above mentioned field tests, five devices tested were approved by the committee to satisfy these qualifications mentioned earlier.

5. TAP WATER TEST

To verify performance of the aeration devices, particular types of devices were tested using tap water. Those tested were the conventional type plate diffuser (maximum pore size 400 °), jet aerator, and Hanshin-Aquarator. The experiments were done to manifest oxygen transfer efficiency for two kinds: Tap water only and tap water with added surfactant.

5.1 Experimental Equipment and Method

The experimental aeration tank at the Public Waters Research Institute was used for the experiment (see Fig. 8). The capacity of the tank is 200 m^3, and the depth of the water is 5.5 m. The sides and bottom of the tank are made of glass, which allows visual observation through the glass.

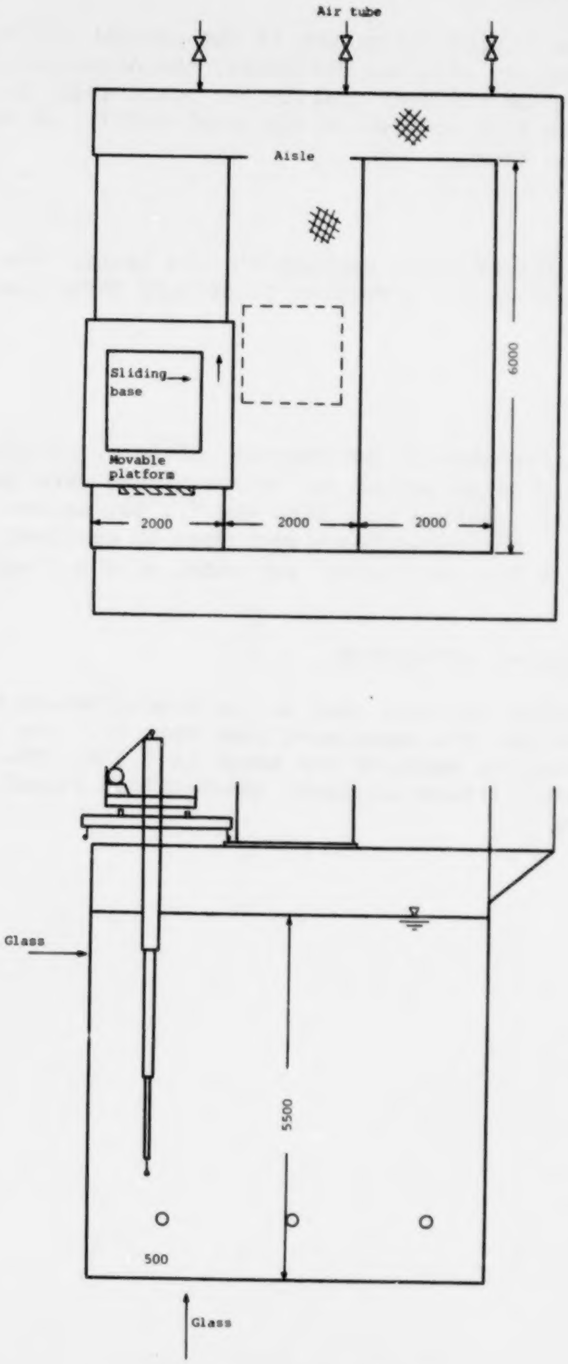

Air tube
Aisle
Sliding base
Movable platform
2000
2000
2000
6000
Glass
5500
500
Glass

Fig. 8 Outline of the experimental tank

Table 8 Amount of chemical additions

Sodium sulfite (Anhyd.)	5 mg/ℓ as MBAS
Sodium sulfite (Anhyd.)	100 mg/ℓ
Cobalt chloride	0.5 mg-Co/ℓ
Surfactant	5 mg/ℓ as MBAS

Sodium sulfite as oxygen remover, cobalt chloride as catalyzer, and detergent (sold on the market) as surfactant were added to tap water for the experiment. Table 8 shows the amount of each that was added. The experiment was performed to measure dissolved oxygen (DO) within the tank (continuous measurement by DO sensor), air flow rate, air pressure, air temperature, and water temperature using a non-steady state method. As for the jet aerator, injected water flow and injected water pressure were measured to estimate the motive power required for water circulation. Concerning the Hanshin-Aquarator, the motive power consumed by the mixing motor was directly measured. Photo. 1 shows a view of the tap water test. Table 9 describes specification of the device used for the experiment.

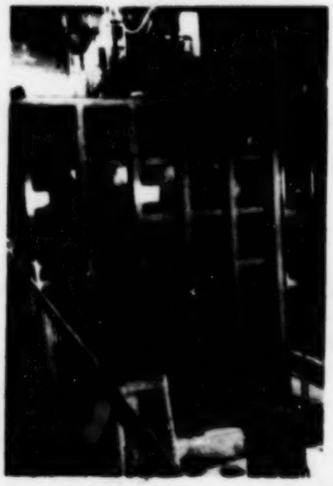


Photo. 1 View of tap water test

From continuous records of DO, the overall oxygen transfer coefficient (Kla) was obtained. Calculation was done using the observed DO saturation method.

Conditions which bring out oxygenation capacity (OC) of 20, 40, 80, and 100 $mg/O_2/\ell \cdot h$ were obtained in a preparatory experiment. The experiment was done under each condition obtained in the preparatory experiment.

Table 9

Plate diffuser	Maximum pore size; 400 μ Permeability (dry air condition); 3,000 cc/min. cm^2 Area rate of diffusers; 8% Depth of plate; 4.5 m, spiral flow type
Jet aerator	Diameter of liquid nozzle; 28 mm Diameter of air-liquid nozzle; 50 mm, 7 nozzles Depth of plate; 4.5 m, spiral flow type
Hanshin-aquarator	Diameter; 1.15 m Draft tube height; 0.65 m Weight; 575 kg Number of installations; 1 unit (installed at the center)

Kla is converted to 20°C using the expression (1) below.

$$\text{Kla (20)} = \text{Kla (T)} \times 1.024^{(T-20)} \quad .... \quad (1)$$

where T = water temperature at the experiment
Kla (T): Kla at T°C

The oxygenation capacity was calculated where DO within the tank was 0 mg/ℓ, the basis on which the oxygen transfer rate (%) and power efficiency (kg-O_2/kWh) were obtained.

5.2 Results of the Experiment

Fig. 9 shows each oxygen transfer rate obtained from the conventional type plate diffuser and jet aerator on the vertical axis of the graph and the corresponding oxygenation capacity on the horizontal axis. The experiment was performed using tap water. Each oxygen transfer rate of the jet aerator and Hanshin-Aquarator is beyond that of the conventional type plate diffuser. The dotted lines indicate the results of the experiment when surfactant was added: The oxygen transfer rate deteriorated 40 - 50% with the conventional type plate diffuser, while such a transfer rate was increased a little in the Hanshin-Aquarator. It seems that this is because there is a difference between the aeration function of each method. Both the jet aerator and Hanshin-Aquarator use, in addition to a blower, a recycling pump or submerged impeller. The power consumed by these should be considered to estimate power efficiency.

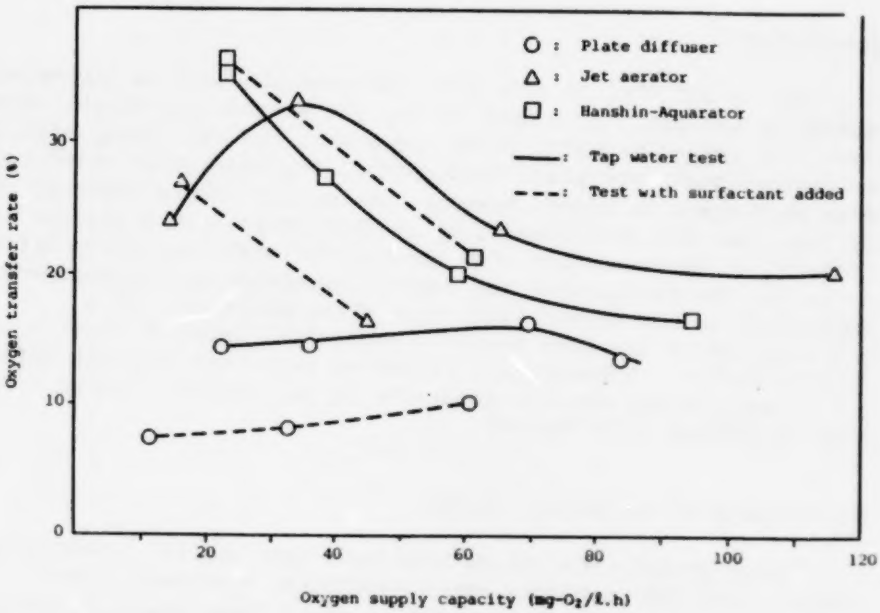


Fig. 9 Comparison of oxygen transfer rates among aeration devices

Fig. 10 shows the comparison of power efficiencies. The tap water test with the three methods mentioned above shows hardly any difference in efficiencies. When surfactant is added, however, the performance of the jet aerator and that of the Hanshin-Aquarator exceed that of the conventional type plate diffuser.

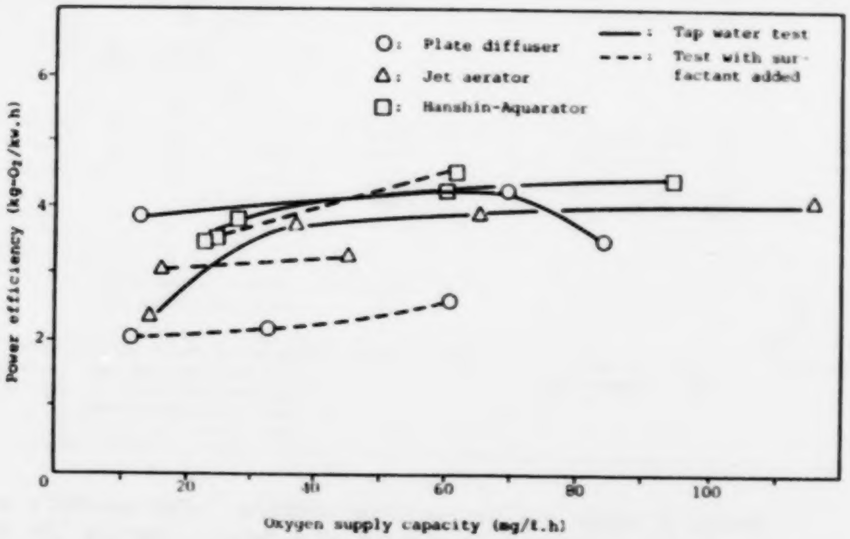


Fig. 10 Comparison of power efficiency among aeration devices

5.3 Conclusion

Fig. 9 and Fig. 10 show that the aeration devices employing forced mixing by external force such as the jet aerator and Hanshin-Aquarator have high oxygen transfer rates and almost the same power efficiencies as conventional type plate diffusers in the experiment using tap water. When surfactant is added, however, the oxygen supply capacity of either of them does not deteriorate, or it deteriorates by a smaller rate compared to deterioration with the conventional type plate diffuser. As a result, the jet aerator and Hanshin-Aquarator are advantageous in comparison to the conventional type plate diffuser. Such characteristics are also found in mechanical aeration devices employing surface mixing. Accordingly, it can be said that the main effect of the jet aerator and/or Hanshin-Aquarator is the oxygen transfer function such as surface film renewal.

6. PERFORMANCE AT THE ACTUAL PLANTS

Those tested aeration devices have been applied practically to twenty-eight (28) publicly owned wastewater treatment plants. Newly established treatment plants tend to adopt these aeration devices. Table 10 lists typical treatment plants employing each device.

Table 10 Examples of operating aeration devices

Device name	Installation	Treating capacity (m^3/day)	Starting date of operation	Energy saving effects	Remarks
Jet Aerator	Hirano Treatment Plant, Osaka-city	29,600	March, 1983	Power efficiency in the aeration tank is 2.5 kg-O_2/kWh. Power consumption per 1 m of treating water is 0.084 kWh/m^3.	Other treatment plants employing the jet aerator have been operating without problems since 1975.
Hanshin Aquarator	Nakahama Treatment Plant, Osaka-city	53,000	November 1978	Results indicated by technical evaluation.	Often used as a denitrification mixer.
Bayer Injector	Kawanakajima Treatment Plant, Mobara-city	12,000	April, 1982	Power consumption was cut by 20%.	Clogging of the nozzle can be avoided by cleaning the circulating water strainer once a week.
Full-surface Aeration by Plate Diffuser	Central Treatment Plant, Hamamatsu-city	246,000	February 1981	Results indicated in technical evaluation.	-
Dome Diffuser	Central Treatment Plant, Nagasaki-city	50,000	June, 1981	Oxygen transfer rate in the aeration tank was increased by 56%. Power consumption was cut by 20%.	Slime is generated partially on the surface of the diffuser.

Photo. 2 shows the jet aerator setting. Jet aerators are mostly set in a deep aeration tank (about 10 m deep). During the operating period of a jet aerator of eight years, no maintenance has been performed on the aeration device itself.

Photo. 3 shows the Hanshin-Aquarator setting and Photo. 4 the operating aeration tank. This device is easy to install because the air pipe and frame are of unitary design. This allows installation (with the help of a wrecker) even while the tank is running. Employment of a draft tube enables setting the aquarator in a deep aeration tank. These days the number of treatment plants that employ biological nitrification and denitrification are increasing, and Hanshin-Aquarators are being adopted for them. This is because the Hanshin-Aquarator can be effectively utilized as a mixer.

Photo. 5 shows an example of the Bayer injector. At this treatment plant, this device is used for denitrification. The device has a small diameter nozzle with a simple strainer. The strainer needs to be cleaned once about a week. No other operations are required for maintenance.

Photo. 6 shows the setting of the full-surface aeration process by the conventional type plate diffuser. This method has been operating without any problems since it was set up in 1981.

Photo. 7 indicates the setting of the dome diffuser. Not only in the dome diffuser but also in the conventional type plate diffuser and tube diffuser, the surface of the plate or tube becomes covered with sludge when used in a mixture of activated sludge. It has not been clarified to what extent the oxygen supply capacity is deteriorated under such conditions.

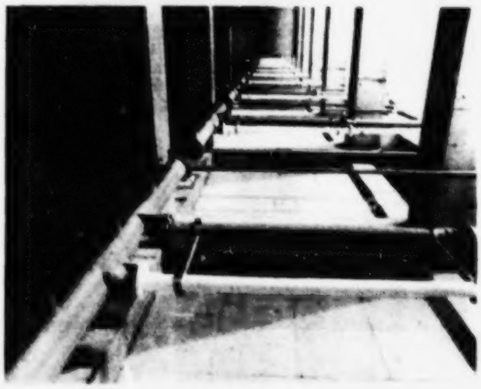


Photo. 2 Jet aerator setting

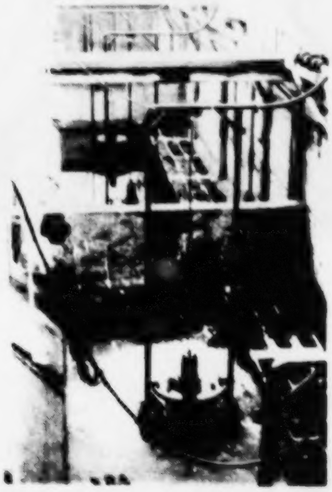


Photo. 3 Hanshin aerator setting

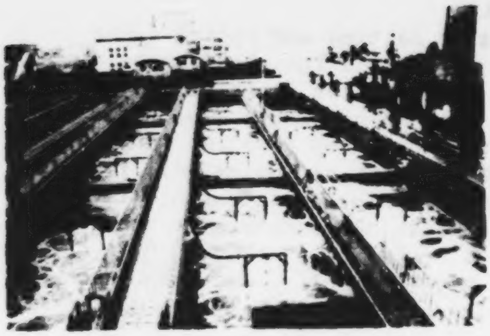


Photo. 4 Aeration tank with Hanshin aerator

Photo. 5 Bayer aerator setting

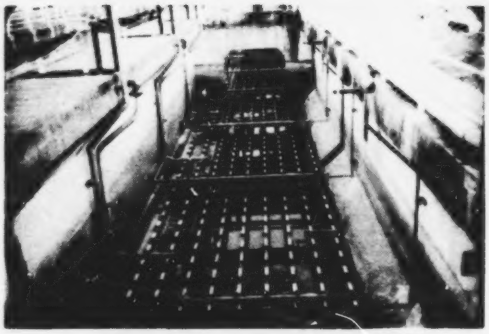


Photo. 6 Plate diffusers of full surface setting

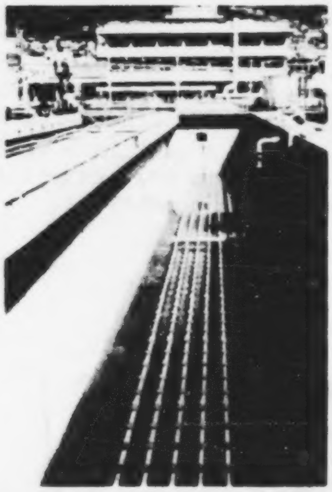


Photo. 7 Dome diffuser setting

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FULL SCALE EVALUATION
OF
BIOLOGICAL PHOSPHORUS AND NITROGEN REMOVAL

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FULL-SCALE EVALUATION OF BIOLOGICAL PHOSPHORUS AND NITROGEN REMOVAL

1. INTRODUCTION

In Japan, eutrophication of lakes or coastal seas has been known for long. But it was not until 1979 that the first definite step against it was taken.

In 1979, the Environment Agency set up guidelines for the reduction of phosphorus discharge into the Seto Inland Sea. Then, effluent standards concerning nitrogen and phosphorus for the Lake Biwa and the Lake Kasumigaura were formulated by the corresponding prefectural governments to administer water quality control with a full commitment.

Later, in December 1982, the Environment Agency promulgated environmental standards concerning nitrogen and phosphorus in lakes.

Table 1 Environmental water quality standards on nitrogen and phosphorus in lakes and reservoirs

Category	Purpose of water use	Standard values	
		Total nitrogen	Total phosphorus
I	Conservation of natural environment, and uses listed in II-V	0.1 mg/ℓ or less	0.005 mg/ℓ or less
II	Water supply classes 1, 2 and 3 (excluding special types); Fishery type 1, bathing; and uses listed in III-V	0.2 mg/ℓ or less	0.01 mg/ℓ or less
III	Water supply class 3 (special types), and uses listed in IV-V	0.4 mg/ℓ or less	0.03 mg/ℓ or less
IV	Fishery type 2, and uses listed in V	0.6 mg/ℓ or less	0.05 mg/ℓ or less
V	Fishery type 3; industrial water; agricultural water; conservation of the living environment	1 mg/ℓ or less	0.1 mg/ℓ or less

Notes: 1. The standards are measured in terms of annual averages.
2. The standards for total phosphorus are not applicable to agricultural water uses.

At present, the designation of lakes to the categories of the environmental standards are being formulated by the hand of the Central Council for Control of Environmental Pollution.

In addition, the Law concerning Special Measures for Conservation of Lake Water Quality was put into effect in March 1985.

According to this new law, which is based on the Water Pollution Control Law, it is provided for that such lakes that need special regulations and overall control measures on account of serious water pollution be designated, and that water quality preservation plans be formulated for such designated lakes in order to: (1) control the pollution loadings from new or expanded sources, (2) control effluents from designated sources, (3) and control the equipment and its use at specified and constructive specified sources.

Of the lakes to be designated, about 40 will have both nitrogen and phosphorus discharge regulation, and for about 1,000 phosphorus discharges will be limited.

Pursuant to the amendment of the Ordinance for the Enforcement of the Water Pollution Control Law, nitrogen and phosphorus effluent standards for facilities discharging 50 m^3/day or more will be put into effect in mid-July, 1985, to limit total nitrogen to less than 120 mg/ℓ. (daily average, 60 mg/ℓ) and total phosphorus to less than 16 mg/ℓ (daily average, 8 mg/ℓ).

Backed by these recent legislative measures, the administration for water quality preservation has made a major step toward the prevention of not only organic pollution but also eutrophication.

2. DEVELOPMENT OF ADVANCED TREATMENT TECHNOLOGIES IN JAPAN, AND ITS BACKGROUND

In 1972, the Public Works Research Institute of the Ministry of Construction installed a 250 m^3/day pilot plant in the Shitamachi Wastewater Treatment Plant, Yokosuka, in order to start the research on the phosphorus removal of the secondary effluent by the coagulation precipitation process, marking the first step in the development of advanced treatment technologies in Japan. To follow suit, the Japan Sewage Works Agency, and major cities including Sapporo, Tokyo, Yokohama, Yokosuka, Kawasaki, Osaka, Kyoto, Kobe, Kita-Kyushu Fukuoka launched their research and development projects for advanced treatment technologies.

These R & D efforts were concentrated on physicochemical processes as represented by filtration, coagulation/precipitation, break-point chlorination, adsorption of ammonia by zeolite, and ammonia stripping.

The achievements of these projects have been presented at the Inter-Government Cooperation Conference on the Advanced Wastewater Treatment Technology sponsored by the Ministry of Construction for lively discussions and information exchange. The information about the advanced wastewater treatment technology from the United States contributed greatly to technology development in this field in Japan, particulary the information brought through the US/Japan conference on Sewage Treatment Technology, which started in 1971.

With the outbreak of Oil Crisis in mid-1971 as a turning point, saving energy and resources has become a social imperative, and has become even more important at energy-guzzling wastewater treatment plants.

Japan's R & D efforts for advanced wastewater treatment technology have naturally been shifted away from energy and resource intensive physicochemical processes toward the implementation of biological nitrogen and phosphorus removal processes.

The Public Works Research Institute of the Ministry of Construction has achieved substantive results in the field study of single-stage nitrified liquor recycled nitrogen removal process, development of a nitrogen and phosphorus removal process in which the nitrified liquor recycled nitrogen removal process and simultaneous coagulation process are combined, and in many other R & D projects.

The Japan Sewage Works Agency has completed a demonstration test of nitrified liquor recycled nitrogen removal process after pilot plant study, and is currently pushing forward a field test on an anaerobic-aerobic process for simultaneous removal of nitrogen and phosphorus.

In recent years, interest has been growing in the development of biological technologies for removal of nutrients, and pilot plant study or full-scale field investigations are under way by many municipalities, including Ibaraki, Nagano, Tokyo, Kawasaki, Yokosuka, Nagoya, Fukuoka and Kyoto. The research and development in this field has also been promoted vigorously by private research institutions.

Triggered by the enforcement of effluent standards for lakes and by the needs of times for the saving of energy and resources, the development and implementation of advanced wastewater treatment technology in Japan have been pushed forward with emphasis on biological processes.

For example, the improvement of the performance of biological secondary treatment process and simple modification of the existing secondary treatment system are actively pursued.

3. IMPLEMENTATION EXAMPLES OF BIOLOGICAL ADVANCED WASTEWATER TREATMENT PROCESSES

3.1 Introduction

In Ibaraki and Shiga Prefectures, the regulations for the control of eutrophication are enforced to protect the water environment of Lake Kasumigaura and Lake Biwa, respectively, and nitrogen and phosphorus removal are carried out at the wastewater treatment plants discharging effluents into these lakes.

As regards nitrogen removal, the single-stage nitrified liquor recycled nitrogen removal process is implemented at the wastewater treatment plants located within the catchment areas of the lakes.

In the small-scale wastewater treatment plants, the endogenous denitrification process is employed when a long retention time in the reactor can be assured. As regards phosphorus removal, the simultaneous precipitation process using alum, poly-aluminum-chloride or other metallic salt as a coagulant in the aeration tank is employed as a practical answer. However, Phosphorus removal by chemical coagulation has drawbacks including high chemical cost and additional production of sludge. As a viable alternative, biological phosphorus removal processes have been the focus of research and development in recent years.

The most stringent effluent standards are enforced in Shiga and Ibaraki, where the T-P concentration in the effluent from sewage treatment plant is limited to less than 0.5 mg/ℓ. This value is considered a target to be achieved by the biological phosphorus removal processes.

Table 2 shows wastewater treatment plants where advanced treatment for nitrogen and phosphorus removal was already in practice as of the end of FY 1984. Their tooling and performance will be discussed in

3.2. The biological phosphorus removal processes are still in the stage of field experimentation by the Public Works Research Institute (Hamamatsu, Kyoto), the Japan Sewage Works Agency (Saitama, Shiga, Itako), Kawasaki Municipal Government and Fukuoka Municipal Government.

In Japan raw sewage is generally weak; the total phosphorus concentration of the effluent of PS usually is in the range of 2 to 6 mg/ℓ.

The phosphorus load of domestic wastewater is taken as 1.8 g-P/capita/day. Assuming that the basic domestic wastewater flow is 300 ℓ/capita/day, the T-P concentration in domestic wastewater is calculated at 6 mg/ℓ.

What posed a serious problem in the experimentation was the influence of rainfalls on the wastewater treatment conditions. In Japan, the Baiu front and typhoons bring heavy rainfalls in summer. Stormwater dilutes the concentration of substrate represented by BOD, etc., while increasing the concentrations of DO and nitrate nitrogen. This is highly detrimental for keeping anaerobic condition in the anaerobic compartment. The role of the anaerobic zone is to feed readily biodegradable substances such as acetic acid to the phosphorus accumulating bacteria through acid fermentation and to help the phosphorus accumulating bacteria to preempt such substances. If stormwater destroys the anaerobic condition, acid fermentation is impaired, and the phosphorus removal performance is degraded.

The biological phosphorus removal process can reduce the phosphorus concentration in the secondary effluent (effluent from the secondary settling tank) to less than 0.1 mg/ℓ in orthophosphate and less than 0.3 mg/ℓ in T-P if the biological phosphorus removal reaction is perfectly controlled by the injection of methanol or by other means. It is generally recognized that, for the ordinary municipal wastewater, the T-P concentration in the secondary effluent can be reduced below 0.5 or 1 mg/ℓ.

Table 2 Wastewater treatment plants practicing advanced treatment for nitrogen and phosphorus removal

Receiving waters	Name of wastewater treatment plant	Design capacity (m^3/day)	Process
Kasumigaura	Kasumigaura Sewage Treatment Plant	55,400	Recycled process, simultaneous precipitation process
	Itako Sewage Treatment Plant	4,200	Recycled process + Simultaneous precipitation process, modified Phoredox process
Lake Hamana	Hitomigaoka Sewage Treatment Plant	1,620	Biological phosphorus removal process + Wuhrmann process
	Kotoh Sewage Treatment Plant	1,980	
Lake Biwa	Konan Chubu Sewage Treatment Plant	14,000	Recycled process + simltaneous precipitation process, modified Phoredox process
	Okinoshima Sewage Treatment Plant	210	Sequential oxidation ditch process + simultaneous precipitation process
	Kosei Sewage Treatment Plant	5,000	Bardenpho process + simultaneous precipitation process

3.2 Implementation Examples

(1) Hitomigaoka Sewage Treatment Plant and Kotoh Sewage Treatment Plant (Lake Hamana)

Both the Hitomigaoka and Kotoh Sewage Treatment Plants stand on Lake Hamana.

The final effluent from the Hitomigaoka and Koto Sewage Treatment Plants must satisfy the strict water quality standard shown in Table 3 in order to prevent the pollution and eutrophication of Shonai Bay and Lake Hamana. Both plants have advanced treatment facilities such as nitrification, denitrification and coagulating sedimentation equipment, rapid filters, and activated carbon absorption towers, in addition to secondary treatment facilities. To cut down the operational and maintenance costs, both plants are also equipped with extended aeration tanks designed to provide a longer retention time for the energy- and resources-saving type biological nitrification-denitrification using the endogenous denitrification reaction, as well as for the phosphorus removing process.

Table 3 Water quality standard in Shonai Bay and Lake Hamana

BOD	5 mg/ℓ
COD	10 mg/ℓ
SS	3 mg/ℓ
T-N	5 mg/ℓ
T-P	1 mg/ℓ

Fig. 1 shows the process flow diagrams. (a) is the flow sheet for the non-circulating type treatment process newly developed on the basis of the recent discovery that when nitrogen is removed efficiently by the endogenous denitrification reaction, the phosphorus release takes place in the second stage of the oxic section and its uptake phenomenon is observed in the aerobic section. (b) is the flow sheet for the anaerobic-aerobic non-circulating treatment process developed to improve the phosphorus removal rate by accelerating its release in the presence of organic subtances in the influent wastewater. This process is also based on the endogenous denitrification reaction.

Table 4 shows the wastewater treatment results of these two processes. The greatest misgiving concerning these processes was in the possible decline in the nitrogen removal rate due to the endogenous denitrification.

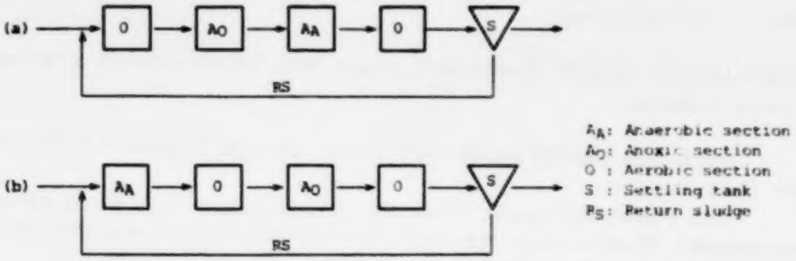


Fig. 1 Flow diagram of Hitomigaoka and Kotoh sewage treatment plant

The phosphorus removal rate is notably different for these two processes. For flow sheet (a), the removal rate is not as high as shown in Table 4 for two reasons: the phosphorus-releasing anaerobic Section is installed after the BOD oxidation and denitrification processes, which makes it difficult to maintain the anaerobic state if the denitrification performance in the preceding line is poor, and the quantity of organic substances is not sufficient to accelerate the phosphorus release. For flow sheet (b), the anaerobic section is installed at the influent inlet, which makes it possible to attain a high removal rate of about 90%.

From the experience gained at these plants, the endogenous denitrification method can be evaluated as a highly useful process of simultaneous nitrogen and phosphorus removal, if the retention time in the aeration tank can be sufficiently extended.

(2) Arakawa Sewage Treatment Plant (Saitama Pref.)

The Arakawa Sewage Treatment Plant is located on the left basin of the Arakawa River which divides Tokyo and Saitama from each other. Its planned design daily maximum flow is 1,380,000 m^3. At present, the facilities with a treatment capacity of 356,000 m^3/day have been completed. The served area is to be convered mostly with a separate sewer system. For the area where sewers have already been installed, however, the combined and separate sewer systems divide almost equally.

The wastewater treatment is carried out according to the conventional activated sludge process.

The sludge treatment flow is as follows.

Sludge thickening tanks → Filter presses → Multiple hearth
Centrifugal thickeners Vacuum filters furnaces

Using one train of the Plant, the Japan Sewage Works Agency conducted an experiment with the nitrified liquor recycled nitrification denitrification process during the 1980 - 81 period and with the biological phosphorus removal process and biological nitrogen and phosphorus removal process during the 1982 - 83 period.

Table 4 Wastewater treatment results by Hitomigaoka and Kotoh sewage treatment plant, Hamamatsu City

Processes	Operating conditions								Water quality							
	Q (m^3/d)	DT (hr)	r (%)	MLSS (mg/ℓ)	BOD-SSL (kg/kg/d)	SRT (d)	Temp (°C)	SVI (mℓ/g)	BOD_5	COD_{Mn}	SS	T-N	NH_4-N	NO_T-N	T-P	Alkalinity
(a)	874	O:A_0:A_A:O- 9.0:9.0: 9.0:9.0	r-69	3440	0.031	38	23.0	270	160/7.1	50/7.0	110/12	31/4.0	20/0.4	0.3/2.0	5.4/1.8	140/57
	805	O:A_0:A_A:O- 9.9:9.9: 9.9:9.9	r-75	4030	0.021	112	16.5	240	140/8.2	53/7.3	98/12	34/6.2	23/0.4	0.4/4.0	4.3/1.9	140/53
(b)	473	A_A:O:A_0:O- 4.9:9.8: 4.9:4.9	r-76	4180	0.059	44	16.0	227	250/12	59/8.0	270/4	31/6.9	18/4.6	0.5/1.3	4.9/0.3	120/69
	656	A_A:O:A_0:O- 3.4:6.8: 5.1:1.7	r-73	3590	0.099	18	23.5	275	260/6.0	60/5.3	240/4.5	45/5.4	23/0.4	0.2/3.4	5.4/0.6	104/55

The Arakawa Sewage Treatment Plant is the first plant in Japan that has implemented these processes.

Table 5 shows design factors of the experiment facilities. When operated as a conventional activated sludge process, the experiment facilities have a daily maximum design flow capacity of 17,600 m^3/day.

Table 5 Design factors of the experiment train

	Width (m)	Length (m)	Effective water depth (m)	Number of tanks	Volume (m^3)	Remarks
Primary settling tank	4.3	50.0	4.0	2	1,720	
Aeration tank	9.0	85.0	5.0	1	3,825	Quadri-sected by baffle walls
Secondary settling tank	4.3	56.0	4.45	2	2,143	

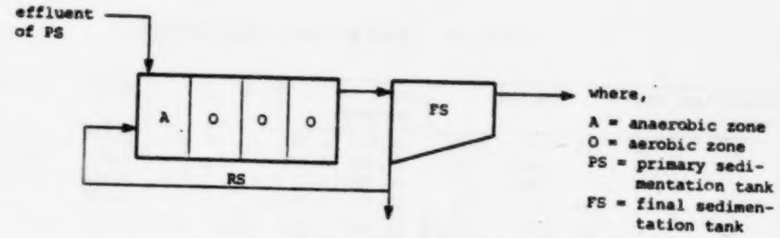
Return sludge is pumped back to the inlet of the aeration tank. The sludge pump is capable of circulation on continuous duty the sludge equivalent of up to about 50% of daily maximum flow.

The aeration tank is divided into four equal compartments with baffle walls. All the diffusers were removed from the first two sections. Instead, the first section was equipped with two 11 kW agitators, and the second section with two 7.5 kW agitators. The agitators in the second section were designed so they could also be used as diffusers.

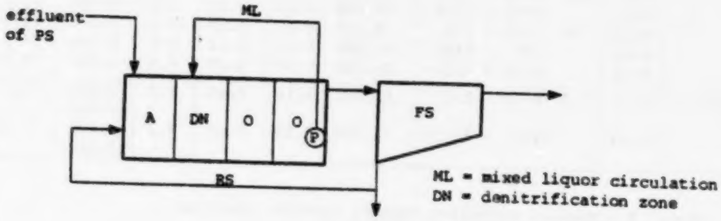
An 11 kW submerged pump was installed near the outlet of the aeration tank to feed nitrified liquor back to the denitrification tank.

As shown in Fig. 2 the experiment was carried out in three operation modes: Mode I in which the first compartment was maintained in anaerobic condition for biological phosphorus removal; Mode II in which the modified Phoredox process was used for the removal of both phosphorus and nitrogen; and Mode III in which the modified Phoredox process was combined with step inflow process for stabilization of biological phosphorus removal.

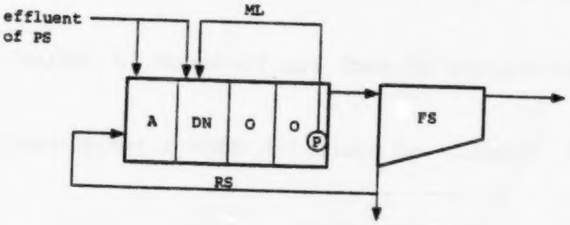
Each operation mode was divided into several runs depending on the operation conditions. (See Table 6.)


effluent of PS
A
O
O
O
FS
RS
where,
A = anaerobic zone
O = aerobic zone
PS = primary sedimentation tank
FS = final sedimentation tank

(a) Operation mode I


effluent of PS
ML
A
DN
O
O
P
FS
RS
ML = mixed liquor circulation
DN = denitrification zone

(b) Operation mode II


effluent of PS
ML
A
DN
O
O
P
FS
RS

(c) Operation mode III

Fig. 2 Operation modes

Table 6 Operation conditions

Operation mode	RUN	Period	Qin (m^3/day)	r	R	T	MLSS	VSS	SVI	t
I	F-1	'82 6/7 - 6/28	22,600	0.15	-	4.1	2,040	72.1	76	21.6
	F-2	6/29 - 7/14	25,300	0.17	-	3.6	2,330	74.4	72	22.1
	F-3	7/15 - 7/30	16,900	0.14	-	5.4	1,940	72.7	74	22.8
	F-4	'83 1/24 - 2/9	18,400	0.16	-	5.0	2,540	76.1	82	14.4
II	F-5	'82 8/27 - 10/25	13,900	0.18	0.59	6.6	1,970	68.7	81	22.4
	F-6	11/8 - 11/24	15,700	0.20	0.50	5.8	3,740	71.1	98	19.2
	F-7	11/25 - 1/19	12,600	0.22	0.68	7.3	4,050	72.3	92	16.2
	F-8	'83 6/22 - 7/8	12,700	0.19	0.64	7.2	3,670	68.4	126	21.4
	F-9	7/13 - 8/12	12,800	0.19	0.64	7.2	2,720	67.3	85	24.4
	F-10	'84 1/25 - 3/5	9,100	0.36	1.25	10.1	3,020	78.7	223	12.3
	F-11	3/9 - 3/23	10,200	0.27	0.51	9.0	2,760	77.5	157	13.3
	F-12	3/26 - 4/13	10,300	0.34	0.90	8.9	2,530	76.9	147	14.9
III	F-13	'83 8/19 - 10/7	17,200	0.14	0.48	5.3	2,360 (3,400)	72.1	90	23.5
	F-14	11/2 - 12/14	12,500	0.19	0.65	7.3	2,660 (3,460)	76.0	138	17.6

Where, T = Nominal retention time of reaction tank (h)
VSS = Organic content of MLSS (%)
SVI = Sludge volume index after 30 minutes (mℓ/g)
and the value of ( ) is MLSS concentration of anaerobic tank
t = Water temperature (°C)

The performance of each run is shown in Tables 7 through 9.

Table 7 Summary of operation mode I performance

(Unit: mg/ℓ)

	F-1		F-2		F-3		F-4	
	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
Temperature (°C)	21.6 (20.5 - 22.1)	-	22.1 (21.2 - 23.3)	-	22.8 (21.8 - 23.2)	-	14.4 (14.0 - 14.9)	-
T-P	3.43 (1.75 - 4.8)	0.44 (0.14 - 0.94)	4.78 (2.28 - 10.3)	0.41 (0.14 - 0.85)	3.30 (1.58 - 4.13)	0.80 (0.17 - 1.24)	4.16 (3.32 - 5.07)	0.80 (0.61 - 1.18)
Soluble PO_4-P	1.57 (0.83 - 2.58)	0.30 (0.05 - 0.70)	2.05 (0.92 - 4.25)	0.23 (0.02 - 0.76)	1.71 (0.74 - 2.57)	0.63 (0.05 - 1.06)	2.13 (1.78 - 2.43)	0.46 (0.27 - 1.0)
T-N	21.6 (14.5 - 26.8)	13.2 (9.9 - 16.1)	29.1 (17.9 - 43.4)	14.6 (12.8 - 16.6)	19.5 (17.9 - 20.7)	11.3 (10.2 - 12.2)	31.8 (30.5 - 34.6)	20.7 (19.1 - 22.6)
NH_4^+-N	15.0 (10.0 - 17.2)	5.5 (3.7 - 8.0)	13.9 (10.7 - 16.7)	10.9 (5.4 - 13.0)	(4.1 - 12.8)	1.4 (1.0 - 2.2)	20.8 (19.8 - 22.3)	17.4 (14.2 - 20.3)
BOD	87 (53 - 113)	6.0 (4.1 - 7.5)	106 (72 - 162)	5.3 (4.3 - 7.3)	74 (67 - 79)	4.2 (2.1 - 5.0)	116 (101 - 131)	6.7 (5.8 - 7.5)
Soluble BOD	30.6 (14.6 - 47.5)	1.6 (0.8 - 4.3)	32.9 (19.2 - 45.1)	1.0 (0.8 - 1.2)	24.5 (23.1 - 27.2)	4.2 (0.4 - 7.4)	57.2 (51.3 - 64.2)	3.7 (1.8 - 8.7)
SS	71 (40 - 120)	4.5 (1.8 - 11.4)	109 (30 - 260)	3.1 (1.5 - 4.6)	62 (24 - 130)	2.0 (0.7 - 3.7)	83 (48 - 114)	9.2 (4.6 - 13.8)
M-Alkalinity	127 (102 - 144)	72 (58 - 90)	139 (115 - 154)	125 (103 - 148)	131 (91 - 146)	66 (49 - 95)	151 (146 - 156)	145 (134 - 153)
P/MLSS (%)	4.09 (3.69 - 4.49)		3.91 (3.60 - 4.25)		3.98 (2.75 - 5.08)		3.82 (3.65 - 4.10)	

Notes: 1. Upper line shows the average and lower line shows (minimum - maximum)
2. P/MLSS was measured with the sample taken at the end of nitrification zone.

Table 8 (a) Summary of Operation mode II performance

(Unit: mg/ℓ)

	P-5		P-6		P-7		P-8	
	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
Temperature (°C)	22.4 (18.6 - 25.1)	-	19.2 (19.0 - 19.5)	-	16.2 (14.3 - 17.8)	-	21.4 (19.9 - 22.8)	-
T-P	2.16 (1.11 - 4.93)	0.52 (0.09 - 1.13)	2.57 (1.85 - 3.67)	0.19 (0.14 - 0.23)	4.35 (2.50 - 6.71)	0.28 (0.21 - 0.53)	5.18 (4.20 - 7.50)	1.39 (0.00 - 2.39)
Soluble PO_4-P	0.79 (0.33 - 1.59)	0.37 (0.01 - 0.95)	0.94 (0.45 - 1.41)	0.05 (0.03 - 0.07)	2.13 (0.82 - 3.66)	0.11 (0.05 - 0.24)	2.47 (1.48 - 3.38)	1.01 (0.00 - 1.61)
T-N	16.5 (8.0 - 22.9)	8.3 (4.8 - 10.7)	22.1 (16.1 - 29.7)	10.6 (7.7 - 13.6)	28.1 (17.2 - 34.8)	11.8 (7.8 - 17.1)	24.1 (17.5 - 31.8)	10.2 (5.9 - 15.5)
NH_4^+-N	10.0 (3.0 - 14.6)	0.2 (0.0 - 2.0)	14.4 (9.6 - 19.6)	5.6 (0.0 - 10.1)	17.7 (9.5 - 20.9)	4.9 (0.0 - 8.2)	14.3 (7.2 - 18.3)	0.9 (0.0 - 3.6)
NO_T^--N	0.5 (0.0 - 1.9)	7.2 (3.5 - 9.6)	0.2 (0.02 - 1.1)	4.0 (2.3 - 6.9)	0.5 (0.02 - 1.7)	5.6 (3.5 - 7.3)	0.3 (0.02 - 1.0)	6.7 (4.9 - 9.3)
BOD	60 (28 - 112)	2.7 (1.4 - 10.3)	71 (53 - 95)	4.5 (1.1 - 6.6)	100 (45 - 130)	4.0 (1.6 - 6.4)	63 (26 - 96)	3.5 (2.2 - 5.4)
Soluble BOD	16.1 (6.8 - 35.3)	0.9 (0.2 - 1.7)	31.7 (17.5 - 41.9)	0.9 (0.3 - 1.2)	40.5 (22.1 - 53.0)	0.9 (0.4 - 1.7)	19.6 (12.7 - 26.7)	1.5 (0.8 - 2.3)
SS	55 (20 - 144)	3.5 (0.5 - 19)	60 (38 - 72)	3.0 (0.3 - 4.4)	94 (42 - 150)	3.7 (0.1 - 13)	61 (26 - 90)	2.3 (0.4 - 5.6)
M-Alkalinity	123 (58 - 147)	66 (37 - 76)	146 (129 - 157)	101 (62 - 121)	147 (105 - 158)	93 (72 - 110)	131 (55 - 154)	71 (60 - 86)
P/MLSS (%)	3.52 (2.88 - 4.75)		3.62 (3.42 - 3.77)		4.04 (3.60 - 4.70)		4.40 (3.81 - 4.68)	

Notes: 1. Upper line shows the average and lower line shows (minimum - maximum)
2. P/MLSS was measured with the sample taken at the end of nitrification zone.

Table 8 (b) Summary of Operation mode II performance

(Unit: mg/ℓ)

	P-9		P-10		P-11		P-12	
	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
Temperature (°C)	24.4 (22.0 - 27.2)	-	12.3 (10.5 - 13.0)	-	13.3 (12.7 - 13.9)	-	14.9 (13.8 - 16.0)	-
T-P	5.07 (3.06 - 9.45)	1.29 (0.57 - 2.06)	3.02 (1.43 - 3.67)	0.21 (0.09 - 0.39)	2.95 (1.99 - 3.69)	0.20 (0.11 - 0.27)	2.66 (2.32 - 3.12)	0.16 (0.13 - 0.22)
Soluble PO_4-P	2.03 (1.59 - 3.68)	0.65 (0.27 - 1.13)	0.98 (0.71 - 1.32)	0.05 (0.01 - 0.21)	0.99 (0.76 - 1.36)	0.03 (0.02 - 0.04)	0.83 (0.49 - 0.94)	0.02 (0.01 - 0.03)
T-N	20.3 (15.9 - 24.7)	9.7 (7.8 - 11.0)	29.1 (24.6 - 35.6)	10.8 (7.7 - 13.6)	33.2 (23.4 - 45.3)	14.7 (11.5 - 17.4)	27.8 (21.5 - 32.6)	11.8 (8.7 - 13.8)
NH_4^+-N	14.3 (10.9 - 16.6)	0.1 (0.0 - 0.3)	16.5 (12.6 - 20.6)	7.5 (3.7 - 10.9)	17.2 (13.4 - 20.9)	10.4 (7.0 - 15.0)	17.0 (12.4 - 20.3)	7.8 (2.4 - 10.5)
NO_T^--N	0.1 (0.02 - 0.2)	8.5 (6.9 - 9.8)	0.8 (0.09 - 2.1)	1.5 (0.5 - 3.0)	1.1 (0.3 - 1.9)	1.9 (0.7 - 3.4)	0.4 (0.1 - 0.8)	2.8 (1.7 - 4.4)
BOD	99 (36 - 211)	2.0 (1.5 - 3.1)	118 (86 - 176)	3.8 (1.5 - 6.7)	133 (105 - 182)	5.6 (2.9 - 8.9)	88 (22 - 114)	5.3 (3.0 - 6.9)
Soluble BOD	40.3 (10.7 - 55.2)	1.0 (0.7 - 2.0)	47.5 (29.8 - 66.4)	0.9 (0.4 - 2.0)	45.9 (28.6 - 65.6)	1.2 (0.6 - 2.3)	44.5 (31.8 - 58.5)	0.7 (0.5 - 0.9)
SS	56 (26 - 124)	1.3 (0.2 - 3.6)	76 (61 - 122)	3.2 (1.8 - 4.9)	74 (41 - 102)	3.6 (1.5 - 6.0)	77 (58 - 90)	3.7 (2.6 - 6.0)
M-Alkalinity	141 (115 - 155)	67 (55 - 75)	143 (127 - 151)	117 (85 - 136)	140 (123 - 158)	123 (100 - 135)	136 (108 - 149)	115 (93 - 132)
P/MLSS (%)	4.22 (2.81 - 5.34)		2.87 (2.35 - 3.33)		2.56 (2.39 - 2.86)		2.50 (2.35 - 2.90)	

Notes: 1. Upper line shows the average and lower line shows (minimum - maximum)
2. P/MLSS was measured with the sample taken at the end of nitrification zone.

Table 9 Summary of operation mode III performance

(Unit: mg/ℓ)

	F-13		F-14	
	Influent	Effluent	Influent	Effluent
Temperature (°C)	23.5 (19.2 - 26.0)	-	17.6 (16.0 - 19.0)	-
T-P	2.02 (1.09 - 4.07)	0.25 (0.16 - 0.56)	3.08 (1.69 - 4.47)	0.24 (0.10 - 0.71)
Soluble PO_4-P	0.62 (0.29 - 1.40)	0.10 (0.04 - 0.28)	1.10 (0.65 - 2.08)	0.05 (0.01 - 0.20)
T-N	15.7 (6.5 - 21.8)	8.7 (6.4 - 10.4)	26.9 (17.9 - 34.5)	7.9 (5.0 - 10.1)
NH_4^+-N	10.0 (2.9 - 14.9)	1.7 (0.0 - 5.3)	14.1 (5.1 - 21.3)	1.3 (0.0 - 2.9)
NO_T^--N	0.5 (0.05 - 4.2)	6.1 (3.1 - 9.3)	0.2 (0.09 - 1.3)	5.9 (2.7 - 7.9)
BOD	82 (20 - 231)	3.3 (1.5 - 5.4)	103 (66 - 130)	3.7 (1.7 - 6.4)
Soluble BOD	27.2 (9.2 - 56.9)	1.0 (0.5 - 2.5)	44.3 (22.7 - 62.3)	0.8 (0.5 - 1.5)
SS	42 (14 - 89)	1.8 (0.0 - 3.2)	59 (22 - 79)	1.5 (0.2 - 2.5)
M-Alkalinity	131 (76 - 162)	90 (67 - 115)	136 (103 - 194)	76 (49 - 104)
P/MLSS (%)	3.32 (2.21 - 5.42)		3.58 (2.46 - 6.69)	

Notes: 1. Upper line shows the average and lower line shows (minimum - maximum)

2. P/MLSS was measured with the sample taken at the end of nitrification tanks.

In Japan, while the DO concentration and the NO_T-N concentration of inflow rise under the effect of heavy rainfalls during the rainy and typhoon seasons, the substrate concentration and the oxygen consumption rate of activated sludge fall due to an increase in flow rate. As a result, it becomes difficult to maintain anaerobic conditions in anaerobic zone.

F-5 without step inflow and F-13 with step inflow were performed almost in the same period of heavy rainy searson though in different years, with regard to phosphorus and nitrogen removal efficiency. Their nitrogen removal efficiency is almost the same, considering the larger inflow rate of F-13. Although the T-P concentration of inflow seems the same as shown in Fig. 3, the phosphorous removal efficiency of F-13 with step inflow operation, is much more stable as shown Fig. 4. In F-13 the average T-P concentration of effluent and the average T-P removal efficiency reached 0.25 mg/ℓ and 88%, respectively, while they were 0.52 mg/ℓ and 76%, in F-5.

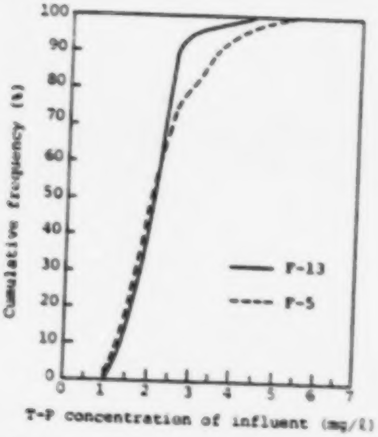


Fig. 3 Cumulative frequency curve of influent T-P concentration

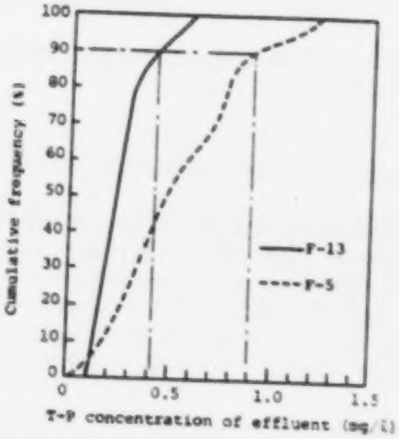


Fig. 4 Cumulative frequency curve of effluent T-P concentration

COD of influent and the T-P concentration of effluent in both F-5 and F-13 are also shown in Fig. 5. The effect of rainfall appears as a drop in COD of influent. While in F-5 the T-P concentration of effluent increased in many days when the substrate concentration was low, the T-P concentration of effluent in F-13 was kept in a low level and all the measurements were below 0.5 mg/ℓ except for a few days after the experiment started.

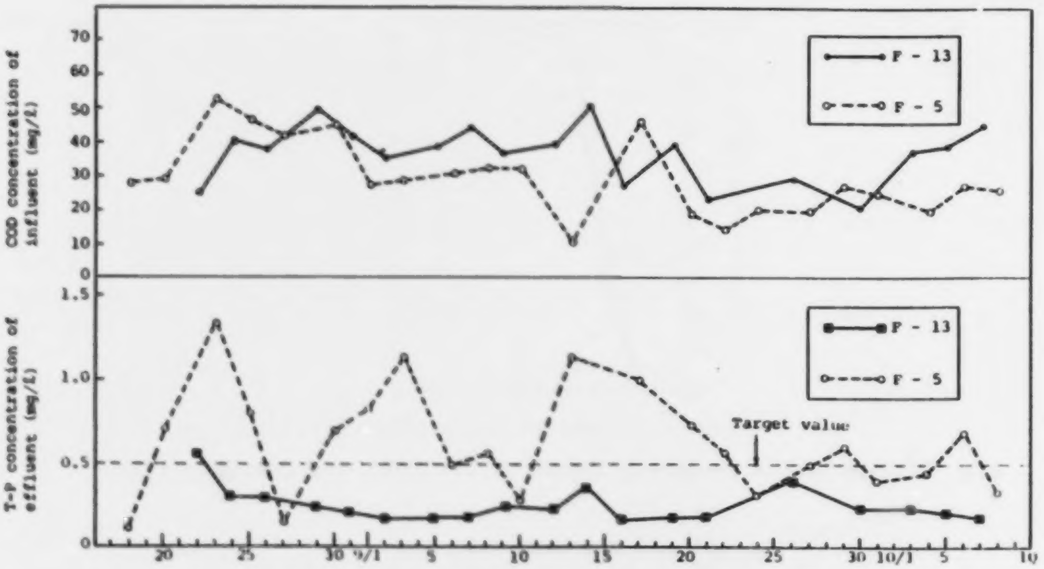


Fig. 5 Daily variation in COD concentration of influent and T-P concentration of effluent

(3) Iriezaki Sewage Treatment Plant (Kawasaki City)

The Iriezaki Sewage Treatment Plant handles sewage collected from the central part of Kawasaki City, and has been in operation since 1961. The served area presents a swale lying between the Tama River and the Bay of Tokyo, and stormwater is drained by pumping. Wastewater is also pumped to the Iriezaki Sewage Treatment Plant. The sewage collection is made by the combined sewer system.

The design capacity of the Plant is 360,000 m^3/day. At present, the facilities with a daily capacity of 300,000 m^3 have been completed.

The wastewater is treated according to the conventional activated sludge process, and the sludge treatment system is composed of gravity thickeners, belt press filters and rotary kilns.

At the Iriezaki Plant, the supernatant from the gravity thickeners and night soil are discharged just upstream of the primary settling tanks, and the characteristics of influent to be treated are subjected to a larger change than when the raw sewage alone is handled.

The schematic view and design particulars of the east facility, part of which is used as the test train, are given in Fig. 6 and Table 10, respectively.

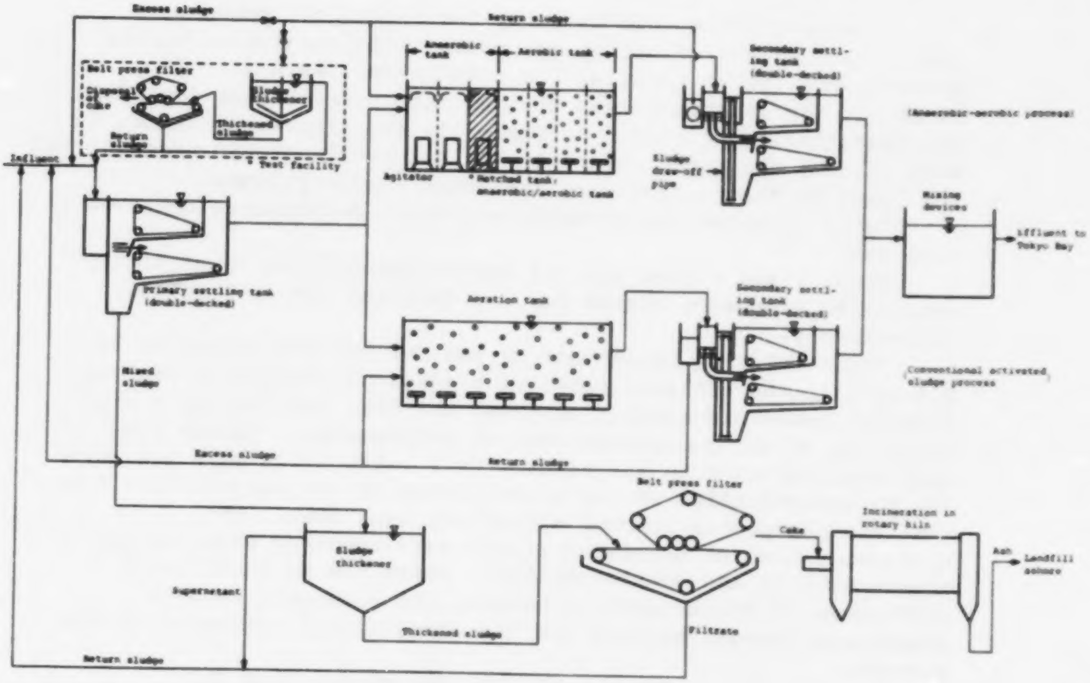

Excess sludge
Return sludge
Belt press filter
Disposal of cake
Sludge thickener
Thickened sludge
Return sludge
* Test facility
Influent
Anaerobic tank
Aerobic tank
Agitator
* Hatched tank: anaerobic/aerobic tank
Secondary settling tank (double-decked)
Sludge draw-off pipe
(Anaerobic-aerobic process)
Mixing devices
Effluent to Tokyo Bay
Primary settling tank (double-decked)
Aeration tank
Secondary settling tank (double-decked)
(Conventional activated sludge process)
Mixed sludge
Excess sludge
Return sludge
Belt press filter
Incineration in rotary kiln
Cake
Ash
Landfill ashore
Sludge thickener
Supernatant
Thickened sludge
Filtrate
Return sludge

Fig. 6 Flow sheet of the east train of Iriezaki Sewage Treatment Plant

Table 10 Design particulars of the test facility

Process		Item Name of facility	Width (m)	Length (m)	Effective water depth (m)	Number of tanks	Volume (m^3)	Retention time (hrs.)	Overflow rate (m^3/m^2/day)
Anaerobic-aerobic process	15°C up	Anaerobic tank	9.0	53.2	6.5	1 tank 3 (compartment)	3,112.2	2.5	-
		Aerobic tank	9.0	83.7	6.5	1 tank 3 (compartment)	4,396.4	3.9	-
	below 15°C	Anaerobic tank	9.0	32.95	6.5	1 tank 2 (compartment)	1,927.6	1.5	-
		Aerobic tank	9.0	103.95	6.5	1 tank 4 (compartment)	6,081.0	4.9	-
	Secondary settling tank		8.6	71.5	3.0	3 tanks	6,127.5	4.9	16.3
Conventional activated sludge process		Aeration tank	9.0	136.9	6.5	1 tank	8,008.6	6.4	-
		Secondary settling tank	8.6	71.5	3.0	3 tanks	6,127.5	4.9	16.3

Note: The retention time and overflow rate refer to 30,000 m^3/day.

An anaerobic-aerobic facility for biological phosphorus removal and another facility for conventional activated sludge process were run in parallel for comparison. As stated in Table 11, the experiment was divided into several runs depending on the operating conditions. In Table 11, two figures are shown in each column; the upper refers to the anaerobic-aerobic process, and the lower to the conventional activated sludge process.

Table 12 shows the phosphorus removal performance achieved in each run.

Figs. 7 and 8 show the T-P and orthophosphate concentration cumulative frequency curves for the influent and effluent, respectively.

The cumulative percentage of the T-P concentrations of up to 0.5 mg/ℓ in the effluent from the biological phosphorus removal process (anaerobic-aerobic process) was 96%, bearing out a high stability of the phosphorus removal performance. It was when the MLSS concentration in the reactor fell below 2,000 mg/ℓ and when the DO concentration in the mixed liquor of the aerobic tank fell that the phosphorus removal efficiency went down. The characteristics of generated sludge were compared with respect to thickenability and dewaterability. There was no significant difference in thickenability between the anaerobic-aerobic phosphorus removal process and the conventional activated sludge process.

As regards the dewaterability, the sludge from the anaerobic-aerobic process was found 1.5 to 2% lower in cake water content than that from the conventional activated sludge process.

Table 11 Test conditions

Item \ Test period	I 1983 (Apr. 11 - Jun. 10)	II-1 (Jun. 11 - Aug. 16)	II-2 (Aug. 17 - Oct. 5)	III (Oct. 6 - Nov. 13)	IV-1 (Nov. 14 - Jan. 17, 1984)	IV-2 (Jan. 18 - Feb. 29)
Influent flow (m^3/day)	35,400	41,500	43,200	36,000	31,500	35,900
	-	39,000	40,200	35,300	30,300	35,200
Air ratio (times)	2.3	2.1	1.3	1.6	2.6	2.5
	-	2.2	2.1	2.0	2.9	2.9
Return sludge flow (m^3/day)	9,230	10,200	12,400	11,400	8,810	10,300
	-	13,750	13,330	13,370	13,440	13,050
Retention time in anaerobic tank (hrs.)	2.1	1.8	1.7	2.1	2.4	1.3
	-	-	-	-	-	-
Retention time in aerobic tank (hrs.)	3.3	2.8	2.7	3.3	3.7	4.1
	-	4.9	4.8	5.4	6.3	5.[illegible]
BOD-SS loading (kg/kg/day)	0.15	0.16	0.17	0.15	0.18	0.17
	-	0.16	0.17	0.14	0.16	0.30
T-P-SS loading (g/kg/day)	11.60	12.30	11.70	8.10	9.19	7.70
	-	12.20	15.60	7.15	7.83	13.20
Secondary settling tank overflow reate (m^3/m^2/day)	19.2	22.5	23.4	19.5	17.1	19.5
	-	21.1	21.8	19.1	16.4	19.1
Retention time in secondary settling tank (hrs.)	4.2	3.5	3.4	4.1	4.7	4.1
	-	3.8	3.7	4.2	4.9	4.2
VSS/SS (%)	68.2	62.9	62.3	64.8	73.9	74.7
	69.3	63.1	59.1	61.7	75.0	75.7
T-P/SS (%)	3.92	3.56	3.46	3.22	3.66	3.5[illegible]
	3.82	3.32	2.88	2.56	3.02	2.95
SVI (mg/ℓ)	125	138	76	117	200	214
	93	86	58	77	165	141
MLSS (mg/ℓ)	3,570	3,160	2,460	2,390	2,650	3,[illegible]0
	1,640	2,050	1,300	1,550	1,870	1,360
Return sludge concentration (mg/ℓ)	7,440	10,900	8,750	7,210	6,790	8,910
	5,020	6,930	4,150	3,370	3,670	3,770
Aerobic tank water temperature (°C)	19.6	22.3	23.7	19.5	15.7	12.3
	19.6	21.8	23.[illegible]	19.5	15.7	12.2
SRT (days)	8.2	5.0	5.2	8.0	7.1	6.5
	-	3.8	3.5	5.4	5.4	2.9
SAT (days)	5.0	3.1	3.2	4.9	4.4	4.9
	-	3.8	3.5	5.4	5.4	2.9

Note: The upper values in the boxs refer to the anaerobic-aerobic process and the lower values to the conventional activated sludge process.

Table 12 Phosphorus removal performance

Item / Test period	T - P (mg/ℓ)			O - P (mg/ℓ)		
	Raw sewage	Effluent		Raw sewage	Effluent	
		Anaerobic-aerobic process	Conventional activated sludge process		Anaerobic-aerobic process	Conventional activated sludge process
I (Apr. 11 - Jun. 10, 1983)	4.12	0.41 (90.0)	0.73 (82.3)	2.38	0.25 (89.5)	0.57 (76.1)
II-1 (Jun. 11 - Aug. 16)	4.45	0.24 (94.6)	0.91 (79.6)	2.75	0.13 (95.3)	0.65 (76.4)
II-2 (Aug. 17 - Oct. 5)	3.23	0.23 (92.9)	1.04 (67.8)	2.02	0.13 (93.7)	0.75 (62.9)
III (Oct. 6 - Nov. 13)	2.31	0.22 (90.5)	0.72 (68.8)	1.38	0.13 (90.6)	0.58 (58.0)
IV-1 (Nov. 14, 1983 - Jan. 17, 1984)	3.61	0.27 (92.5)	0.88 (74.2)	2.23	0.14 (93.7)	0.65 (70.9)
IV-2 (Jan. 18 - Feb. 29)	3.91	0.28 (92.8)	0.92 (76.5)	2.23	0.15 (93.3)	0.66 (70.9)

Note: The values in parenthese in the effluent column show the removal rate (%).

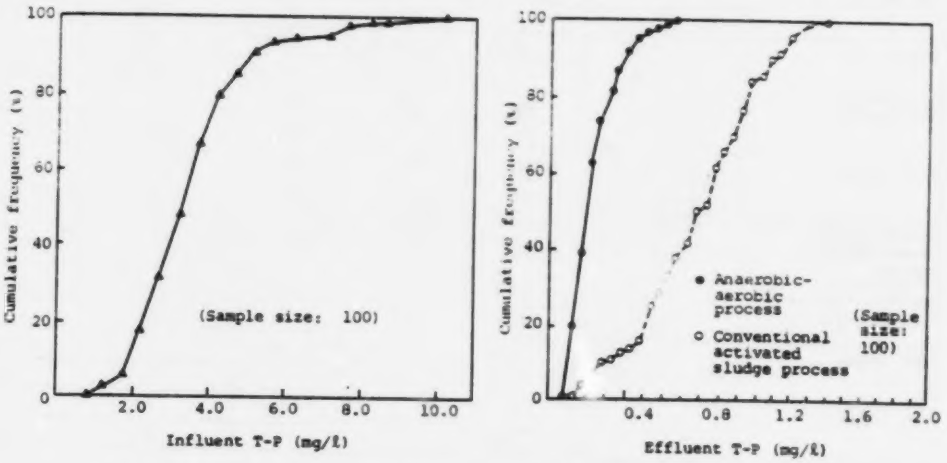


Fig. 7 T-P concentration cumulative frequency curve

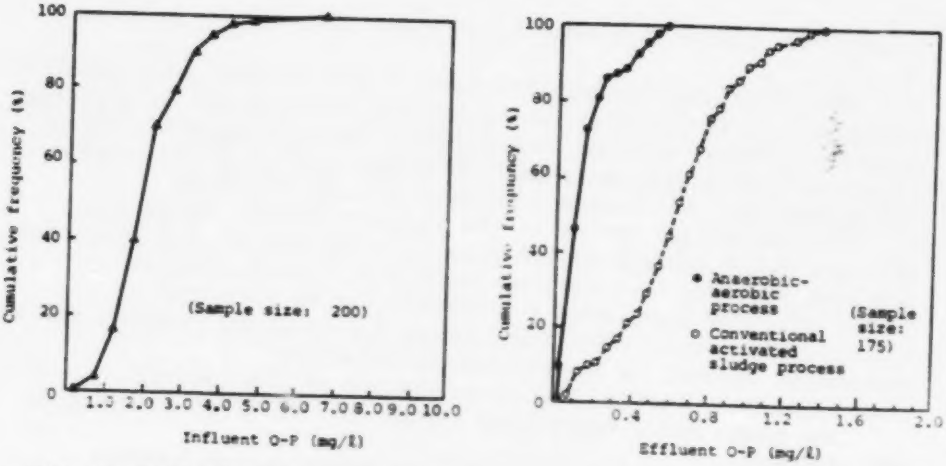
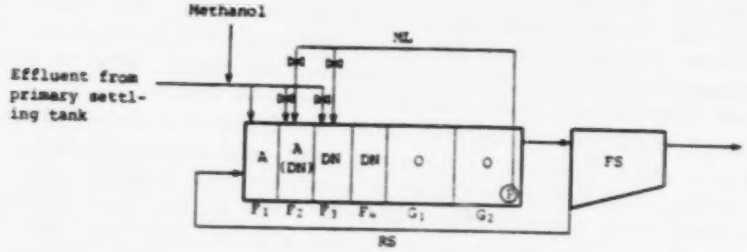
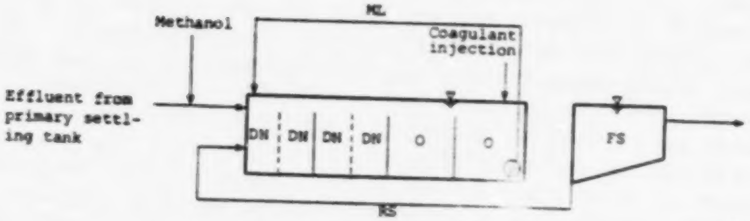


Fig. 8 Orthophosphate P (O-P) concentration cumulative curve


Methanol
ML
Effluent from primary settling tank
A
A (DN)
DN
DN
O
O
FS
P₁
P₂
P₃
P₄
G₁
G₂
RS

(a) Test train


Methanol
ML
Coagulant injection
Effluent from primary settling tank
DN
DN
DN
DN
O
O
FS
RS

(b) Control train

Fig. 9 Flow diagram of Konan Chubu Sewage Treatment Plant (Lake Biwa)

(4) Konan Chubu Sewage Treatment Plant (Lake Biwa)

The Konan Chubu Sewage Treatment Plant was put into operation in April 1982 as Japan's first plant operating on the single-stage nitrified liquor recycled nitrification denitrification process. It serves the southeastern part of Shiga, including the capital of the prefecture, and covers a planned area of 25,500 ha.

The collection system is the separate sewer. While its ultimate daily maximum design flow is 1,020,000 m^3, the Plant has now been operated on a limited scale with a daily flow of 21,000 m^3. The plant uses a nitrified liquor recycled nitrification denitrification process for nitrogen removal, and the phosphorus removal is carried out by injecting poly-aluminum-chloride (PAC) into the end of the reactor.

The plant has a rapid sand filter for the final cleaning of the effluent.

On the other hand, the sludge treatment system used is composed of gravity thickeners for thickening process and filter presses for dewatering process.

At the Konan Chubu Sewage Treatment Plant, one train of facilities has been used for experiment with a biological nitrogen and phosphorus removal process since June 1984. Fig. 9 shows a process flow of the test train and control train.

Each train has a reactor with a volumetric capacity of 2,250 m^3. The reactor is divided into four compartments. In the test train, the first two compartments of the reactor are subdivided into two each in order to make either one eighth or a quarter of the reactor anaerobic and also to measure the denitrification rate in the denitrification chamber with a passable accuracy. The baffles in the reactor for the test trains are constructed to protect surface flow for the purpose of determining the nitrification and denitrification rates accurately. In the control train, however, the top ends of baffles are set about 30 cm below the overflow surface, and a considerable secondary circulation is noticed across the baffle between the denitrification and nitrification compartments. The test train is designed to allow step inflow of raw sewage for the purpose of stabilizing biological phosphorus removal against stormwater inflow.

For accurate determination of influent flow, a self-priming pump is used to pump up the settled sewage from the primary settling tank and feed it through a flowmeter at a constant rate.

At the Konan Chubu Sewage Treatment Plant, the primary settling tank is operated to provide a residence time of 0.5 to 1 hour for the purpose of increasing the BOD source to the reactor. Since the start of test, methanol has been injected to make up for a shortage of BOD source. As BOD of the raw sewage has been increasing annually, the methanol injection rate has been on a steady decline. The running conditions and phosphorus removal performance of the test train are as shown in Figs. 10 and 11.

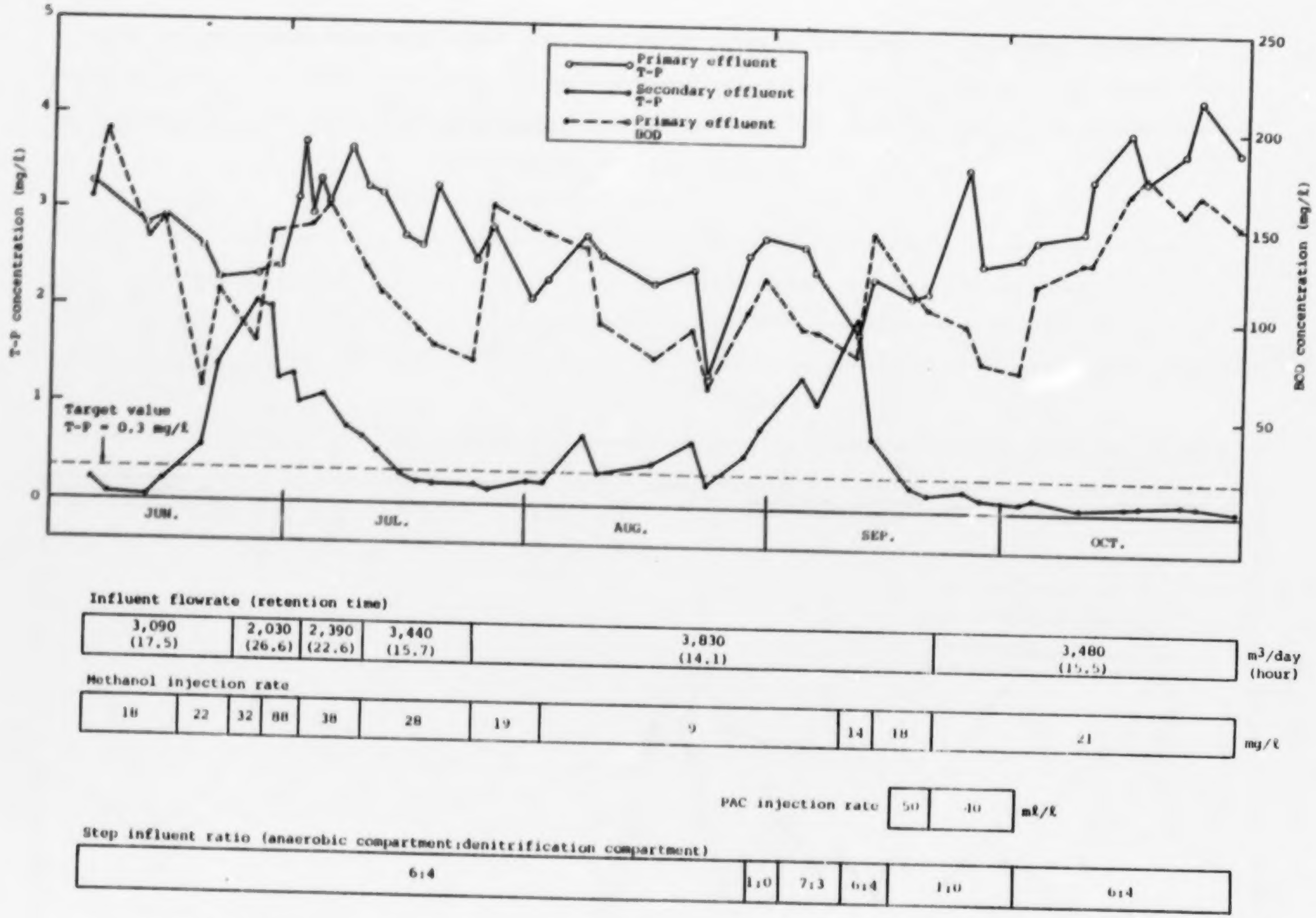


Fig. 10 (a) Operation conditions and phosphorus removal performance (June to October)

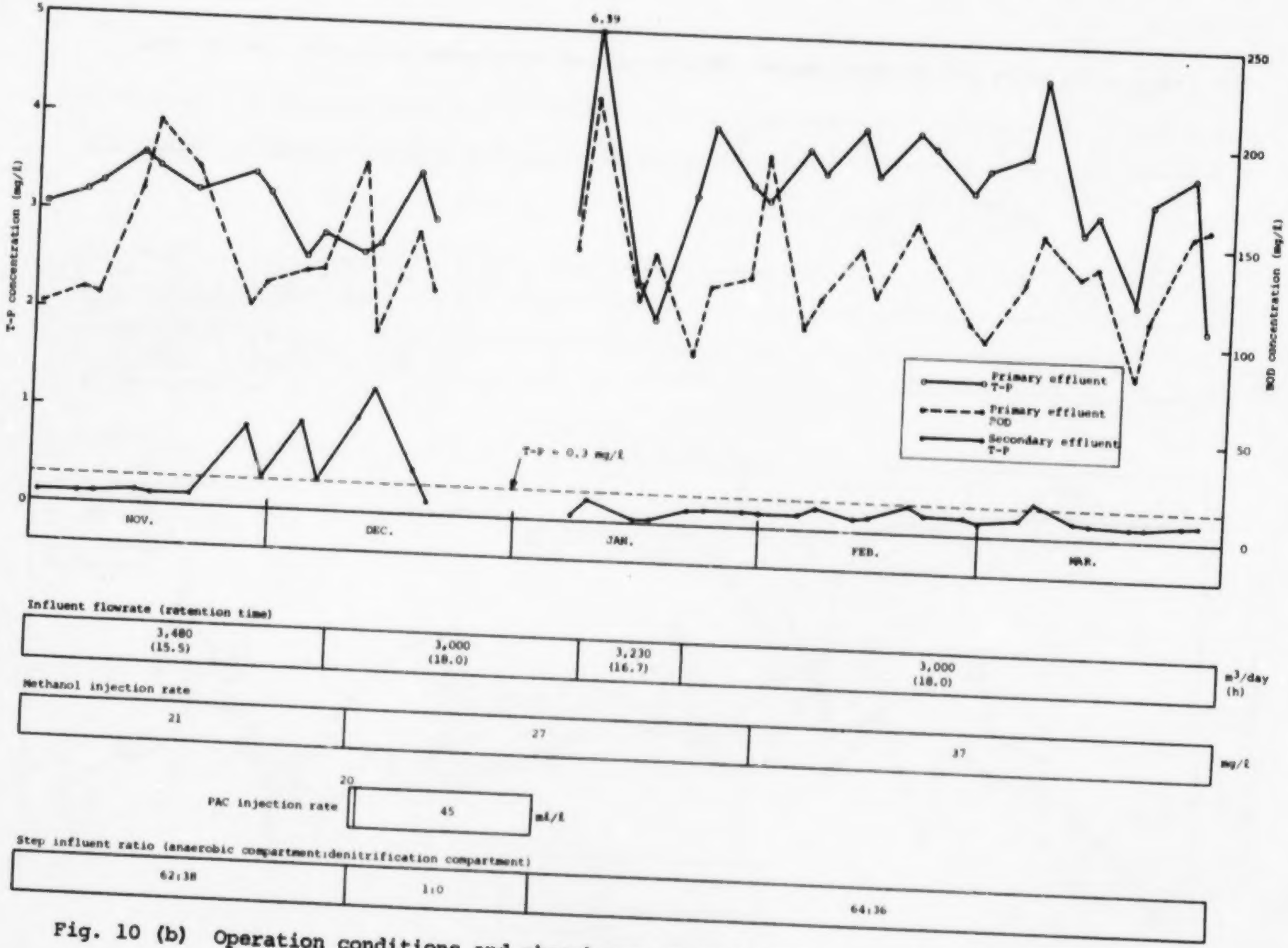


Fig. 10 (b) Operation conditions and phosphorus removal performance (November to March)

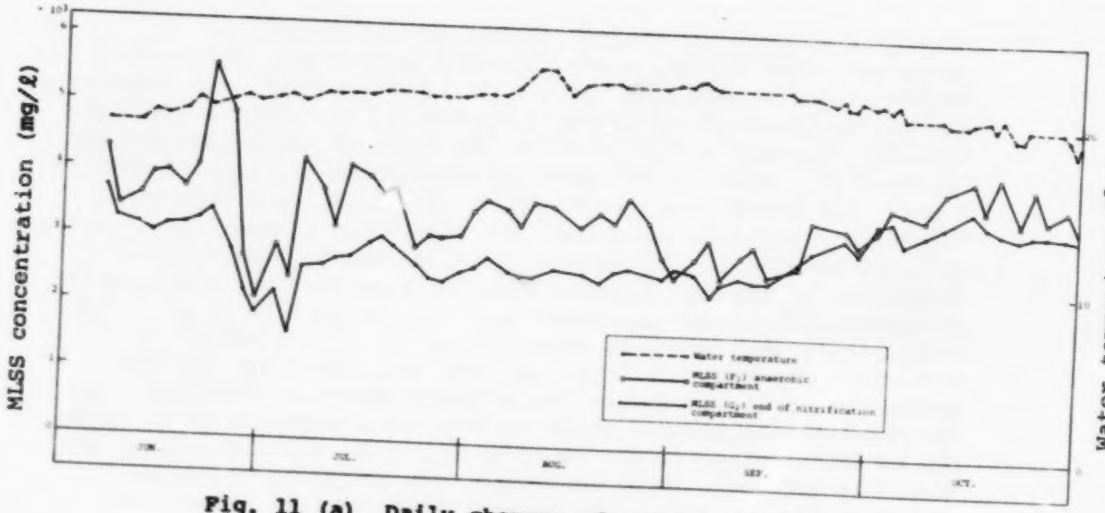


Fig. 11 (a) Daily changes of water temperature and MLSS concentration (June to October)

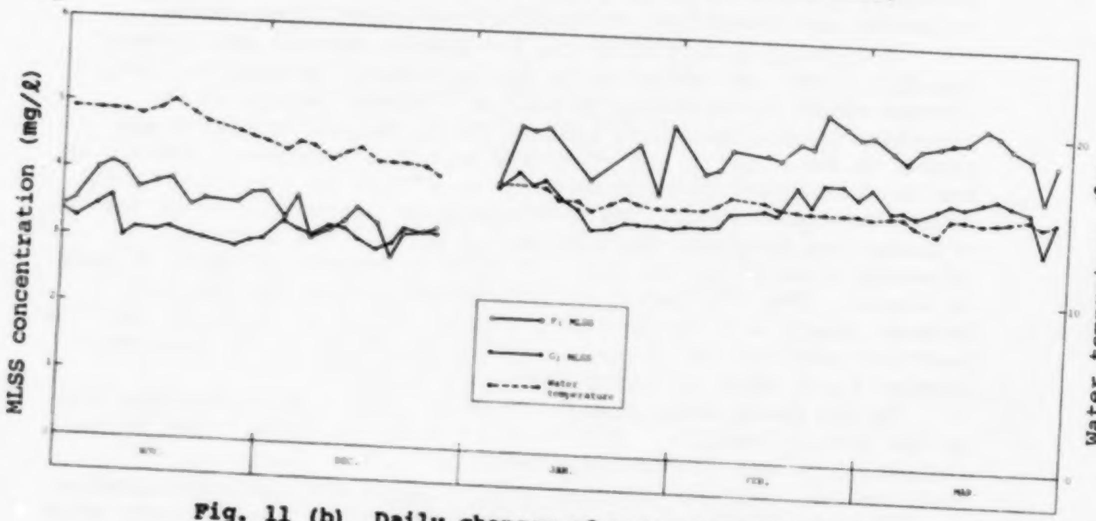


Fig. 11 (b) Daily changes of water temperature and MLSS concentration (November to March)

As the target level of phosphorus in the effluent is set at 0.3 mg/ℓ, the injection of methanol is unavoidable for the moment. It is found from Fig. 8 that the concentration of total phosphorus in the effluent from the secondary settling tank can be held below 0.3 mg/ℓ as far as there is an sufficient supply of BOD. In the early stages of test, sludge from the control train was used as seed sludge, and the effluent quality was very satisfactory owing to the effect of residual coagulant. In about two weeks after start, however, the effluent quality plunged as the residual coagulant was spent. In order to accelerate the growth of biological phosphorus removal sludge and replacement of coagulant-added sludge, heavy injection of methanol and heavy withdrawal of sludge were carried out. As a result, the phosphorus removal performance was improved gradually. In the meantime, the influent flow was increased while the methanol injection rate was decremented. When the methanol injection rate was reduced down to 9 mg/ℓ, the phosphorus concentration in the effluent began to fluctuate. In early September, the phosphorus concentration in the effluent started to rise again. This was due to the overhaul inspection of the self-priming pump for step feed of inflow, which prevented MLSS in the anaerobic zone from being kept at a sufficiently high level as shown in Fig. 8. To overcome the situation, the methanol injection rate was increased, and the phosphorus concentration in the effluent improved gradually. But the plant administrator wanted to keep the phosphorus level below the monthly average target value of 0.3 mg/ℓ, and a coagulant was thus injected for a quick recovery. The coagulant injection continued for two weeks. Transition from chemical removal to biological removal was made without any trouble. From the end of November, phosphorus removal got unstable again, and coagulant injection was restarted, which continued about three weeks.

Following this, biological phosphorus removal was resumed steadily. For the operation of the biological phosphorus removal process which is sensitive to various factors because it is dependent on microbial activities, the coagulant injection was proved to be a useful and effective backup as explained above. As for the biological phosphorus removal process in the Konan Chubu Sewage Treatment Plant, the orthophosphate concentration in the effluent can be stably held below 0.1 mg/ℓ if the methanol injection rate is set at about 20 mg/ℓ in summer and about 30 mg/ℓ in winter. The difference in methanol injection requirement between summer and winter will probably be due to the fact that the reaction rate for the organic acid fermentation in the anaerobic chamber falls when the water temperature is low.

In the Konan Chubu Sewage Treatment Plant, the retention time in the primary settling tank is short, and the BOD/P ratio in the influent into the reactor is relatively high at about 40. Nevertheless, methanol injection is required for the stabilization of biological phosphorus removal process. The volatile fatty acids as a source of readily biodegradable BOD are considered to play an important role in the biological phosphorus removal process. In the Konan Chubu Sewage Treatment Plant, however, about two-thirds of generated sludge is coagulant-injected activated sludge which is fairly stable against biodegradation, and the return load of volatile fatty acids from the sludge treatment system is low. In addition, the retention time in the primary settling tank is short. This makes relatively high apparent BOD due to SS-form BOD which is considered not to work much for the biological phosphorus removal reaction. This reasoning may explain out the high methanol injection requirement even with high BOD/P ratio.

Fig. 12 shows the nitrogen removal performance of the test train and control trains. The target effluent quality is 10 mg•TN/ℓ. In winter when the nitrogen concentration in the influent exceeds 30 mg/ℓ, the methanol injection rate is increased to about 40 mg/ℓ. The total-nitrogen removal efficiency of the nitrified liquor recycled nitrification denitrification process is estimated at 60 to 70% for ordinary municipal wastewater.

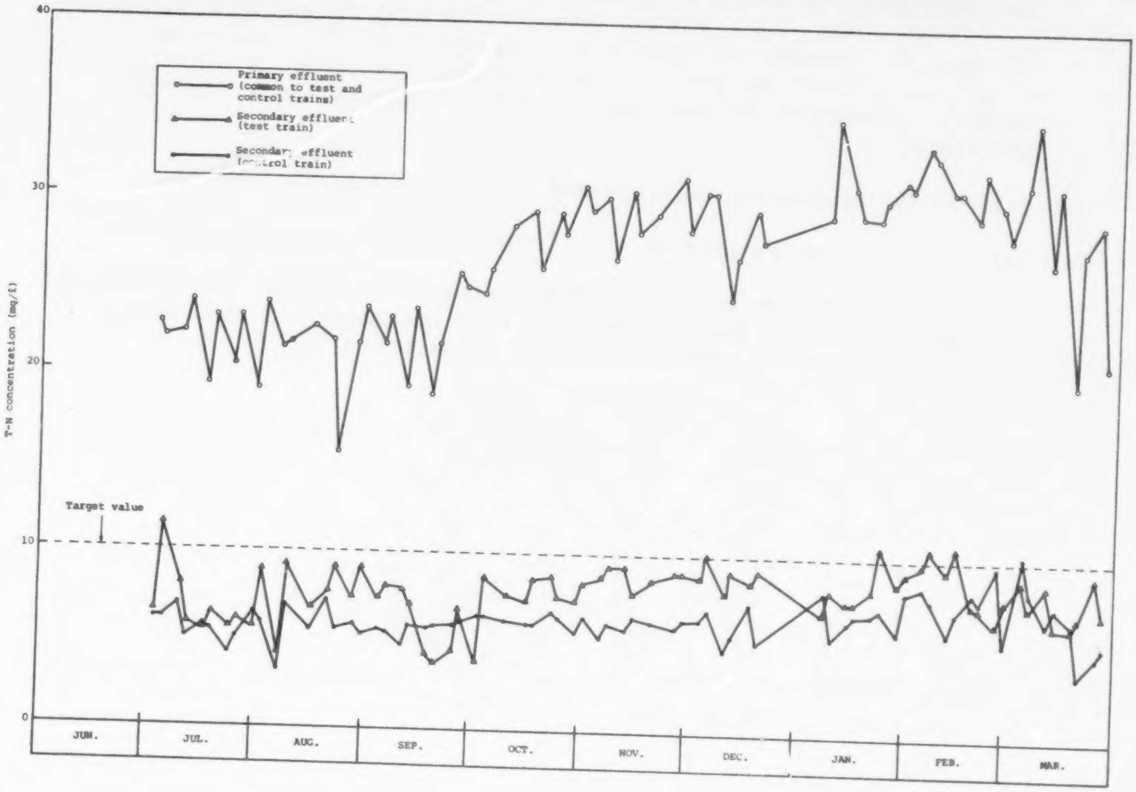


Fig. 12 Nitrogen removal performance

In order to keep the total-nitrogen concentration in the effluent below 10 mg/ℓ even when the nitrogen concentration in the influent exceeds 30 mg/ℓ, it is required to inject methanol to increase the sludge production for the purpose of removing nitrogen in the form of sludge.

The nitrogen concentration in the effluent of the control train was about 2 mg/ℓ lower on the average than that of the test train. This may have been caused by the secondary circulation of nitrified liquor in the control train created by loose partition between the nitrification and denitrification compartments.

4. RECENT TRENDS IN RESEARCH AND DEVELOPMENT

4.1 Introduction

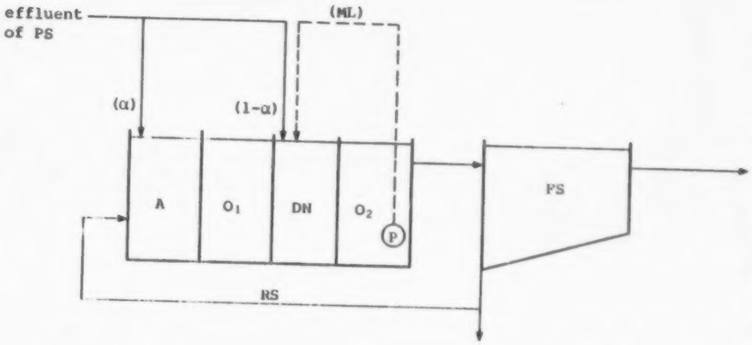
Research and development have been pushed forward through pilot scale plant experiments or bench scale tests in search of the measures to overcome several problems identified as standing in the implementation of advanced biological sewage treatment processes.

Among these, this chapter deals with a step inflow type biological nitrogen and phosphorus removal process designed for stabilized phosphorus removal against stormwater and a process using nitrifying bacteria encapsuled in high polymer gel for improved nitrification efficiency.

4.2 Step Inflow Biological Nitrogen and Phosphorus Removal Process

(1) Operating principles of step inflow process

The biological nitrogen and phosphorus removal efficiency can be improved by feeding influent into the anaerobic zone and denitrification zone in steps as illustrated in Fig. 13.



where,

A = anaerobic zone

DN = denitrification zone

O_1 = the first nitrification zone

O_2 = the second nitrification zone

PS = primary sedimentation tank

FS = final sedimentation tank

ML = mixed liquor recirculation

RS = return sludge

Fig. 13 Proposed biological nitrogen and phosphorus removal process with step inflow

In the step inflow process, the flow of solids brought in by return sludge remains unchanged, but the wastewater inflow into the first chamber of a reaction tank decreases. Therefore, the MLSS concentration in the former part of the reaction tank increases. This feature does good for nitrogen and phosphorus removal as detailed below.

(a) The step inflow process can increase the MLSS concentration in the anaerobic zone without increasing the return sludge ratio, and therefore the inflow of nitrate nitrogen into the anaerobic zone can be minimized. The increase of the MLSS concentration increases the oxygen consumption rate and denitrification rate, eliminating the unfavorable influences of dissolved oxygen and nitrate nitrogen in raw sewage, which pose serious problems during rainy season.

(b) The step inflow process illustrated in Fig. 13 can increase the MLSS concentration in the first nitrification zone. As a result, the total amount of solids that can be held within the entire reactor can be increased considerably. This is particularly useful in winter when the SRT must be extended to make up for the decrease in the growth rate of nitrifying bacteria. If the SRT is to be held constant, the return sludge ratio can be reduced, reducing the evil effect of nitrate nitrogen in the return sludge upon the biological phosphorus removal performance.

(c) In the step inflow process in which the influent is divided into the first chamber, i.e., the anaerobic zone, and the third chamber, viz., the denitrification zone, nitrogen entrained into the first chamber which is nitrified at the second chamber can be removed in the denitrification zone without circulation. In the step inflow process, a considerably high T-N removal can be expected depending on the step inflow ratio.

In the ordinary nitrified liquor recycled nitrification denitrification process, for example, part of nitrogen is removed in the form of sludge, and the ratio of conversion from influent T-N into nitrate nitrogen remains at a level of 60 to 80%. If 20% of T-N in the influent is to be removed in the form of sludge, if the remaining 80% is completely nitrified, and also if nitrate nitrogen returned into the denitrification zone is completely removed, the T-N removal of the nitrified liquor recycled nitrification denitrification process can be expressed by Eq. (1).

$$\text{T-N removal (\%)} = \left(0.2 + 0.8 \times \frac{R}{1 + R}\right) \times 100 \quad \ldots (1)$$

where R = recirculation ratio

$$\left(= \frac{\text{Return sludge flow + Nitrified liquor recirculation flow}}{\text{Influent flow}}\right)$$

In case of the step inflow process, on the other hand, under the same assumptions as before, the T-N removal can be expressed by Eq. (2) on condition that the step inflow ratio is 2:1.

$$\text{T-N removal (\%)} = (0.2 + 0.8 \times (\frac{2}{3} + \frac{1}{3} \times \frac{R}{1 + R})) \times 100 \quad \ldots \text{(2)}$$

These two processes are compared in Fig. 14 with respect to the T-N removal efficiency as a function of the recirculation ratio, R. Compared with the ordinary nitrified liquor recycled nitrification denitrification process, the step inflow process provides a greater T-N removal at a smaller recirculation ratio. This suggests that the nitrified liquor recirculation pump may be dispensed with if the conditions are favorable. Since the step inflow process can increase the T-N removal above 80%, it will be useful, particularly when the effluent standard is tightened or when the nitrogen concentration in the influent rises.

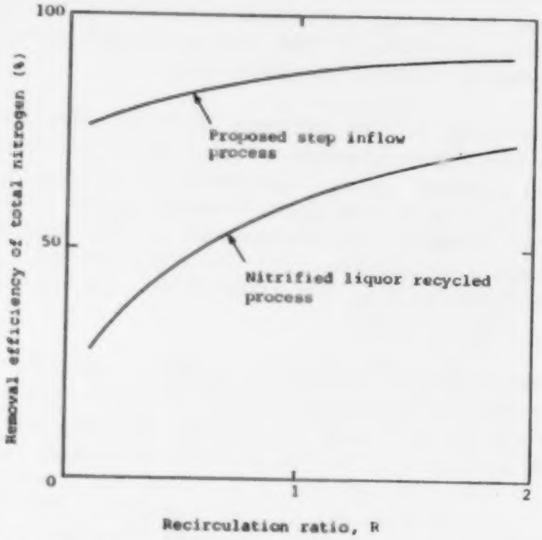
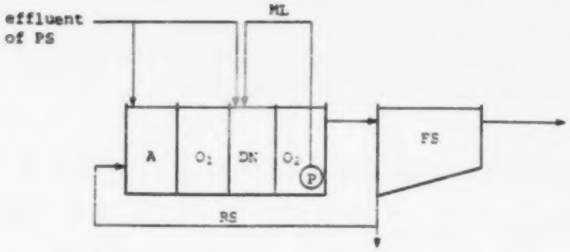


Fig. 14 Relationship between removal efficiency of total nitrogen and recirculatiion ratio R

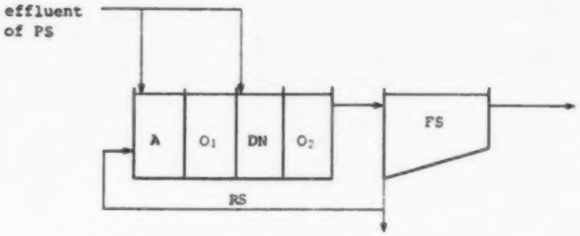
(d) In the step inflow process, the MLSS concentration in the succeeding chambers remains the same as when the step inflow is not applied. Accordingly, it is possible to improve the nitrogen and phosphorus performance without increasing the hydraulic loading and SS loading in the secondary settling tank.

(2) An outline of the experiment

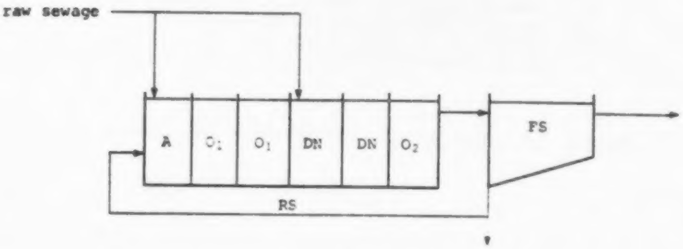
The step inflow process was experimented with using pilot plant (A) where a 9.5 m^3 reactor wad divided into four equal compartments and pilot plant (B) where a 51 m^3 reactor was divided into six equal compartments. The operating modes were as shown in Fig. 15.


effluent of PS
ML
A
O_1
DN
O_2
P
FS
RS

(a) Operation mode IV (pilot plant A)


effluent of PS
A
O_1
DN
O_2
FS
RS

(b) Operation mode V (pilot plant A)


raw sewage
A
O_1
O_1
DN
DN
O_2
FS
RS

(c) Operation mode VI (pilot plant B)

Fig. 15 Operation modes of pilot plants

The raw sewage was pumped from the adjacent sewage treatment plant to the pilot plants. For pilot plant (A), the raw sewage was first run through a primary settling tank. For pilot plant (B), the raw sewage was directly fed into the reactor because of limiting restraints of testing facilities. Influent and effluent were analyzed by making use of 24 hours composite samples.

As regards the water quality in the reactor, spot samples were analyzed at 10:30 a.m. In principle, measurements were carried out three times a week - Monday, Wednesday and Friday.

The process was also tried at a actual sewage treatment plant under the operating conditions specified in Table 6. The operating conditions of pilot plants (A) and (B) are as shown in Tables 13 and 14. The operating period was divided into several runs depending on the operating conditions. The values on the tables are the average values of each run. The influent flow, return sludge flow and nitrified liquor recirculation flow were regulated to be nearly constant in each run.

Table 13 Operation condition of pilot plant A

Operation mode	Run	Period	Qin	r	R	T	MLSS		VSS	SVI	t
							AT2	AT4			
IV	A-1	'84 6/16 - 7/6	24	0.3	0.8	9.5	3350	2320	71.2	112	23.1
	A-2	7/7 - 8/2	24	0.4	0.9	9.5	4010	2880	68.4	121	25.5
	A-3	8/3 - 9/3	24	0.4	0.9	9.5	3670	2700	66.2	119	27.4
	A-4	9/4 - 10/1	24	0.4	0.9	9.5	3600	2790	69.5	132	25.2
V	A-5	'84 10/2 - 10/24	24	0.4	0.4	9.5	3220	2510	71.9	161	22.9
	A-6	11/14 - 12/7	19.2	0.5	0.5	11.9	3780	2990	73.0	256	17.7
	A-7	12/8 - 1/9	15.4	0.5	0.5	14.8	3420	2940	73.1	291	14.8
	A-8	'85 1/10 - 2/1	15.4	0.63	0.63	14.8	4080	3440	75.8	279	14.1
	A-9	2/2 - 3/1	15.4	0.63	0.63	14.8	3690	3240	73.2	291	13.7

where

Qin = Influent flow rate (m^3/day)
r = Return sludge ratio
R = Recirculation ratio
T = Nominal retention time (h)
VSS = Organic contents of MLSS (%)
SVI = Sludge volume index (mℓ/g)
t = Water temperature (°C)

Table 14 Operation condition of pilot plant B

Operation mode	Run	Period	Qin	r	T	MLSS		VSS	SVI	t
						AT3	AT6			
VI	B-1	'84 12/8 - 1/9	79.2	0.5	15.5	4620	3700	75.9	165	14.4
	B-2	'85 1/10 - 2/1	79.2	0.64	15.5	4830	3930	76.3	189	13.4
	B-3	2/2 - 2/22	79.2	0.64	15.5	4490	3650	75.3	202	13.2

(3) Test results and discussions

① Phosphorus removal

Let us express the phosphorus removal performance (Yp) by Eq. (3). Then, its relation to the BOD-SS loading (Xp) in the anaerobic zone which is defined by Eq. (4) is represented by Fig. 16.

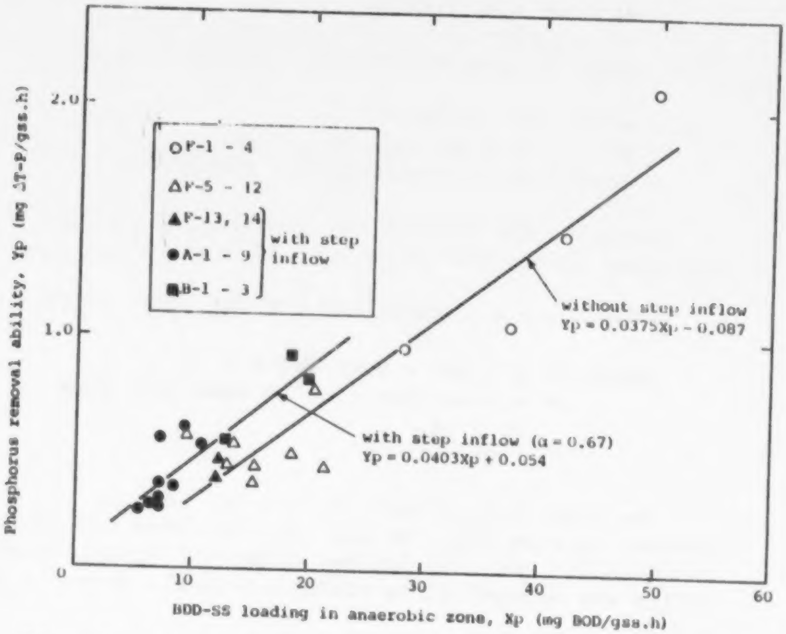


Fig. 16 Relationship between phosphorus removal ability and BOD-SS loading in anaerobic zone

$$Yp = \frac{(Pin - Pout) \times Qin}{MLSSp \times Vp \times 24} \quad \cdots (3)$$

where Yp = phosphorus removal ability (mg ΔT-P/gSS•h);
Pin = T-P concentration in influent (mg/ℓ);
Pout = T-P concentration in effluent (mg/ℓ);
Qin = influent flow (m^3/day);
MLSSp = mixed liquor SS concentration in the anaerobic zone (g/ℓ);
Vp = volume of anaerobic zone (m^3).

$$Xp = \frac{\alpha \times Qin \times BODin}{MLSSp \times Vp \times 24} \quad \cdots (4)$$

where Xp = BOD-SS loading in anaerobic zone (mg-BOD/gSS•h);
α = ratio of inflow to the first compartment.

Eq. (5) representing the line of regression in the figure was determined from the results of the biological phosphorus removal process and modified Phoredox process tried at the actual sewage treatment plant. The result of the step inflow process is given by Eq. (6).

- o Without step inflow:
  $Yp = 0.0375\ Xp - 0.087$ ... (5)
  (correlation coefficient = 0.937, n = 12)

- o With step inflow:
  $Yp = 0.0403\ Xp + 0.054$ ... (6)
  (correlation coefficient = 0.858, n = 14)

Assuming the form of the function $Yp = a \cdot Xp + b$, ΔT-P is expressed by Eq. (7) using Eqs. (3) and (4).

$\Delta T\text{-}P = \alpha \times a \times BODin + b \times MLSSp \times Tp$ ... (7)

where $\Delta T\text{-}P = Pin - Pout$ (mg/ℓ);
Tp = retention time in anaerobic zone
$(= \frac{Vp \times 24}{Qin})$ (h).

By substituting Eqs. (5) and (6) into Eq. (7), we can express ΔT-P by Eqs. (8) and (9), provided that $Tp = 3$ (h), and MLSS for non-step inflow, $MLSSp = 3$ (g/ℓ). MLSS for step inflow was assumed to be $MLSSp = 4$ (g/ℓ), with $\alpha = 0.67$ and $r = 0.3$.

- o Without step inflow:
  $\Delta T\text{-}P = 0.0375 BODin - 0.78$ ... (8)

- o With step inflow:
  $\Delta T\text{-}P = 0.0270 BODin + 0.65$ ... (9)

As shown above, the concentration of total phosphorus removed can be expressed as a function of influent BOD. As shown in Fig. 17, the step inflow process is effective particularly when influent BOD is low.

At influent BOD of 50 mg/ℓ, for example, the step inflow process removes twice as much T-P concentration as non-step inflow process does. It is therefore evident that the step inflow process performs better during summer season when the influent BOD concentration becomes generally low on account of rainfalls, etc. It should be borne mind here that the situation is reversed when the influent BOD concentration exceeds 150 mg/ℓ.

The reason why the step inflow process is better when the BOD concentration is low may be explained as follows.

a. A high MLSS concentration in the anaerobic zone increases the oxygen utilization rate and denitrification rate, which in turn reduces the consumption of BOD necessary for keeping the anaerobic condition.

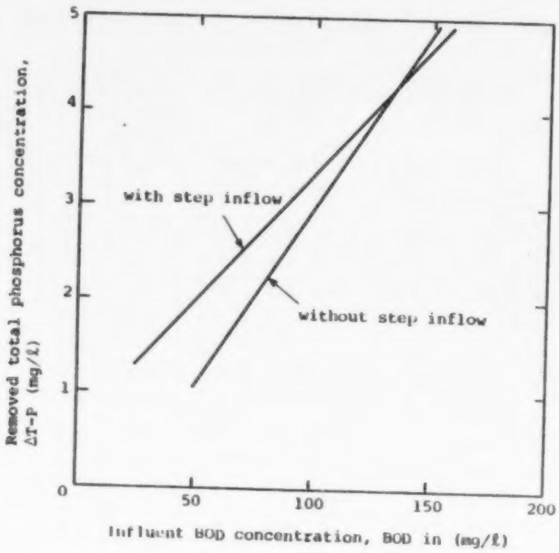


Fig. 17 Relationship between removed total phosphorus concentration and influent BOD concentration

b. The actual residence time in the anaerobic zone is also increased, and the output of readily biodegradable substrate used as culture ground of phosphorus removal bacteria is increased.

② Notrogen removal

a. Increase of SRT

In Run A-6, for example, where step inflow was applied with $\alpha = 0.67$ and $r = 0.5$, the SRT is increased 14% over the non-step inflow process. To maintain the same level of SRT without step inflow, it is necessary to increase the return sludge ratio, r, to 0.61. Although it is supposed for the computation here that the return sludge SS concentration remains unchanged, the fact is that the return sludge SS concentration decreases with increase in the return sludge ratio; namely, that the return sludge ratio must be increased further.

b. Increase of T-N removal efficiency

The nitrogen removal performance of pilot plants (A) and (B) is shown in Table 15.

Table 15 Nitrogen removal efficiency of proposed step inflow process

Operation mode	Run	Influent (mg/ℓ)		Effluent (mg/ℓ)			Denitrification zone (mg/ℓ)		Removal efficiency of T-N (%)
		T-N	NH_4^+-N	T-N	NO_T^--N	NH_4^+-N	NO_T^--N	BOD/N	
IV	A-1	23.2	13.0	7.9	6.4	0.1	3.5	3.7	65.9
	A-2	25.7	11.8	6.2	4.5	0.2	2.1	4.6	75.9
	A-3	24.0	14.1	7.3	4.6	1.1	2.2	3.7	69.6
	A-4	25.7	14.2	7.7	5.0	0.7	2.7	5.1	70.0
V	A-5	28.2	14.3	7.8	6.0	0.4	1.6	6.9	72.3
	A-6	29.3	15.8	10.2	6.0	1.9	2.2	4.8	65.2
	A-7	30.8	16.7	12.8	10.2	1.5	4.8	4.4	58.4
	A-8	35.6	20.6	12.1	5.9	4.6	0.6	9.2	66.0
	A-9	29.5	17.3	11.2	8.9	1.4	4.4	3.5	62.0
VI	B-1	38.0	16.1	5.1	1.5	2.0	0.1	11.0	86.6
	B-2	53.9	20.8	6.1	1.4	2.6	0.3	16.4	88.7
	B-3	47.6	19.1	6.4	4.1	0.9	2.1	11.1	86.6

In the operating mode IV, nitrate nitrogen always stayed on in the denitrification zone. For this reason, the operation of nitrified liquor recirculation pump was cut off in the test thereafter. Assuming that all of nitrate nitrogen running into the denitrification zone is removed, the test conditions dictate that the T-N removal efficiency rate ought to be more than 80% as calculated by Eq. (2). In pilot plant (A), however, the T-N removal rate efficiency 10 to 20% lower than calculated because the BOD/N ratio was generally low as shown in Table 15 to make the denitrification capacity insufficient. Runs A-7 thru A-9 and Runs B-1 thru B-3 were conducted almost concurrently; in pilot plant (B), the raw sewage was directly used, and the T-N removal efficiency became higher than the value calculated from Eq. (2) because the BOD/N ratio got higher.

This is also explained by the fact that, in the primary settling tank, the BOD removal efficiency larger than T-N removal efficiency, and therefore that the BOD/N ratio of influent becomes larger than that of primary effluent.

To sum up, the step inflow process can achieve a T-N removal efficiency of more than 80% without recirculation of nitrified liquor if the BOD/N ratio is high enough to keep a required level of denitrification capacity.

c. Alkalinity in the first nitrification zone

Even though a denitrification zone follows the first nitrification zone, it is almost unlikely that the alkalinity will run short in the first nitrification zone if the step inflow retio is kept at 2:1. During test, the alkalinity of the influent was in the range of 136 to 175 mg/ℓ, and the alkalinity in the first nitrification zone was from 72 to 101 mg/ℓ.

4.3 Improvement of Nitrification Rate by Pelletization of Microbes

One of the recent technological developments is the encapusulation of nitrifying bacteria in high molecular gel such as polyacrylamide. The fixation of nitrifying bacteria in this way prevents the washout of nitrifying bacteria due to shortage of SRT during the low-temperature season, and thus increases the nitrifying efficiency. There are a wide variety of binders available for the fixation of nitrifying bacteria.

The Japan Sewage Works Agency conducted a test for improving the nitrifying efficiency using nitrifying bacteria pelletized with polyacrylamide as binder which has been in wide use for the purpose. The characteristics of the pellets used are shown in Table 16.

Table 16 Initial characteristics of pellets used for the test

	Pellet 1	Pellet 2
Pelletized sludge seed	Activated sludge from recycled process	Ammoniaassimilated activated sludge
Sludge concentration (%)		
Material of binder	Polyacrylamide	Polyacrylamide
Binder concentration (%)	18	18
Pellet size (mm)	3ø x 1.3 - 1.5	2ø x 2
Pellet strength (kg/cm^2)	1.4	1.4
Initial activity* ($mg-O_2/ℓ-gel$)		

* Respiration rate at 20°C, NH_4-N 20 mg/ℓ, and more than DO 4 mg/ℓ.

Fixed microbes of pellet 1 was obtained by directly thickening of activated sludge from the sewage treatment plant operating on the nitrified liquor recycled nitrogen removal process, and that of pellet 2 was obtained by assimilating the same sludge for 2 months using ammonia nitrogen and nutrients alone.

Fig. 18 shows the test arrangement. Two trains of the same test arrangement were used. For one train, pellets of nitrifying bacteria were filled. The other train was not filled with pellets, and was used as a control. The capacity of the reactor was 9 ℓ for pellet 1 and 4 ℓ for pellet 2.

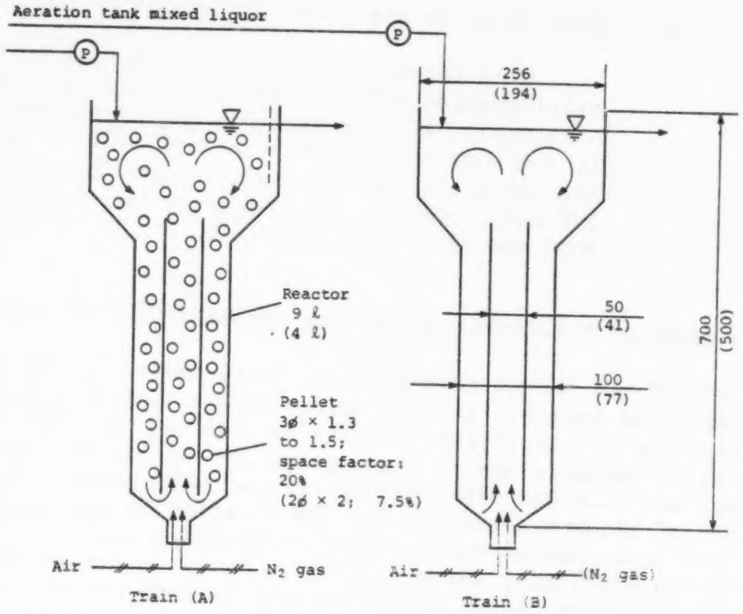

Aeration tank mixed liquor
P
P
256
(194)
Reactor
9 ℓ
(4 ℓ)
Pellet
3ø × 1.3
to 1.5;
space factor:
20%
(2ø × 2; 7.5%)
50
(41)
100
(77)
700
(500)
Air
N₂ gas
Train (A)
Air
(N₂ gas)
Train (B)

Fig. 18 Schematic view of test arrangement

The operating conditions were as specified in Table 17.

Table 17 Operating conditions

Train / Conditions	Case I		Case II	
	Train A	Train B (control)	Train C	Train D (control)
Reactor volumetric capacity (ℓ)	9.0	9.0	4.0	4.0
Inflow rate (ℓ/d)	72	72	19.2-48*	19.2-48*
Retention time (h)	3.0	3.0	5.0-2.0*	5.0-2.0*
DO concentration (mg/ℓ)	2-3	2-3	2-3	2-3
MLSS concentration (mg/ℓ)	1600-2100	1600-2100	2000-2400	2000-2400
Pellet	Pellet 1	-	Pellet 2	-
Pellet space factor (%)	30	0	7.5	0

* The retention time was changed from 5 hours to 3 hours and then to 2 hours according to the increase of the nitrification rate.

In the pellet-using test, the NH_4^+-N concentration in the effluent from the pellet-charged train was lower than that in the effluent from the control train.

As regards the NO_3^--N concentration, the situation was reversed. In train A using Pellet 1, nitrification was more advanced than in train B where suspended sludge alone was used. In the test using Pellet 2, the same tendency was observed.

In the encapusulation of nitrifying bacteria, DO and HN$_4^+$-N are fed through high-molecular gel. Accordingly, the dispersion stage in the gel was suspected to govern the nitrification rate in the low concentration range. This matter was studied, and the results were as shown in Figs. 19 and 20.

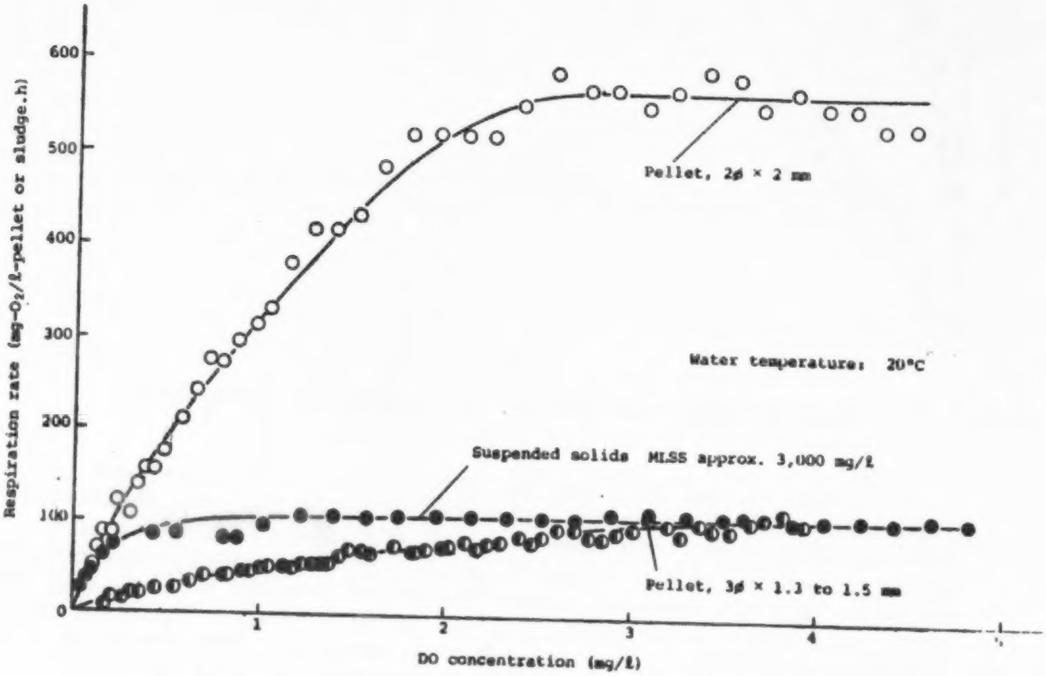


Fig. 19 DO concentration vs. respiration rate

Fig. 19 shows the DO concentration vs. respiration rate relationship with respect to Pellet 1, Pellet 2 and control activated sludge. The control activated sludge was assimilated for about 2 months with NH$_4$Cl as a major substrate. Its MLSS was about 3,000 mg/ℓ. Fig. 20 shows the relationship between DO concentration and respiration rate ratio with the respiration rate at DO = 4 mg/ℓ taken as 100. As shown in Fig. 20 when the nitrifying bacteria were pelletized, the effect of DO on the respiration rate started with a higher concentration range than the activated sludge. This tendency was more conspicuous with Pellet 2 which was larger in size than Pellet 1.

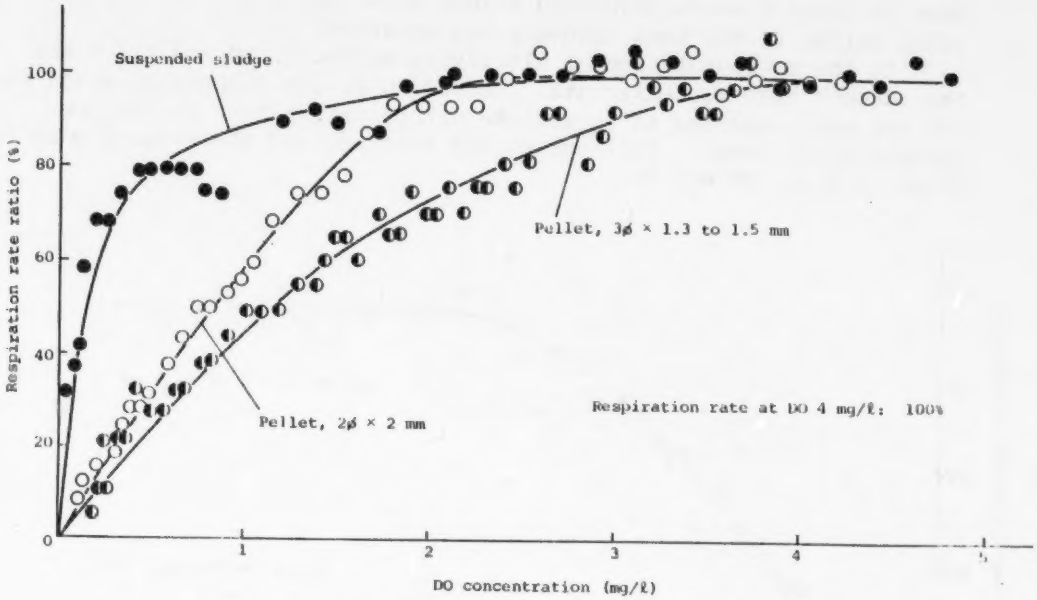


Fig. 20 DO concentration vs. respiration rate ratio

In Fig. 19, Pellet 1 and Pellet 2 show a large difference in respriation rate despite the assimilation under the same conditions. This difference may have come from the difference in the number of bacteria between Pellet 1 and Pellet 2. If the number of pelletized bacteria is small, the activity will not be enhanced sufficiently even after a long period of assimilation. It is therefore important to pelletize high-density sludge of a high activity. The effect of NH$_4^+$-N concentration was also studied; as compared with the activated sludge, there was found no significant influence of NH$_4^+$-N dispersion in the gel used for pelletization on the NH$_4^+$-N removal rate. The findings of the test are summarized as follows.

(1) It is expected that, by using nitrifying bacteria pelletized with polyacrylamide and activated sludge in combination, the nitrifying bacteria can be held at a high concentration level even during a low-temperature for a high-rate nitrification in a short time.

(2) If the initial activity of pellets is low, a required level of nitrification rate may not be achieved for a long period. Microbes may be encrusted upon the surfaces of th pellets depending on the surface property of the pellets. It is therefore highly desirable to establish the pelletizing technology that will ensure a high nitrification efficiency stably for an extended period.

(3) As regards the dispersion of DO and NH_4^+-N through pellets, there is no practical problem. However, the pellets should preferably be as small as possible without detriment to the ease of their handling.

(4) For the implementation of the process, studies must be made on the pellet fluidity conditions, reduction pellet specific gravity, mass production technology of pellet, durability, production cost, operation and maintenance costs, etc.

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PERFORMANCE EVALUATION
OF
OXIDATION DITCH PROCESS

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ENGINEERING EVALUATION OF OXIDATION DITCH

1. CURRENT STATUS OF THE OXIDATION DITCH PROCESS IN JAPAN

1.1 Definition

The oxidation ditch process is one of the variations of the activated sludge treatment process and is generally defined as follows;

(1) An endless channel (called ditch) with a mechanical aerator is used as a reactor.

(2) The mechanical aerator supplies oxygen required for the biological treatment. In addition, the aerator mixes the activated sludge and the wastewater, and provides the flow of the mixed liquid in the ditch, so that the activated sludge is kept in a suspended condition.

(3) The range of BOD-SS loadings is 0.03 to 0.05 kg-BOD/kg-SS/day, the MLSS concentration in the ditch is 3,000 to 5,000 mg/ℓ, and the ditch depth is about 1 to 3 m.

(4) The oxidation ditch process can be operated by two different flow patterns; one is the continuous flow system, and the other is the batch system which uses the ditch as a settling tank as well as a reactor.

1.2 History

The publicly owned wastewater treatment plant employing the oxidation ditch process for the first time in Japan is Yumoto Treatment Plant, in Nikko City. It was put into operation in 1966. Nikko City is very famous for Shogun's shrines, spa and beautiful lakes located in the National Park, and many tourists visit there every year. In early 1960s, the deterioration of the water quality in Lake Unoko brought Nikko City consider treating the wastewater discharged from the hot spring recreation area into the lake. The city made investigation into wastewater treatment systems in various countries considering the climate of Nikko City, where the ambient temperature sometimes falls below -20°C in winter. As a result, the city decided to employ an oxidation ditch process capable of treating wastewater efficiently even in the cold area.

Because the oxidation ditch process was adopted at Nikko City, people become increasingly interested in this process, which resulted in the construction of two wastewater treatment plants employing this process, one in Ogata Village in Akita Prefecture and the other in Toyohashi City. The former was put into operations in 1969 and the latter in 1977. These two plants are located in rural communities. Takane Wastewater Treatment Plant in Toyohashi City treats the wastewater with especially high nitrogen content compared with general domestic wastewater because it includes livestock waste. This characteristics of the wastewater resulted in an operational problem, which was a deficiency of alkalinity due to consumption by nitrification. The mechanical aeration system was replaced in 1981, then nitrogen removal by nitrification and denitrification was intended in the oxidation ditch. As a result, this problem was solved, and the plant continued satisfactory operations since then.

In 1979, the Yufutsu wastewater treatment plant in Tomakomai City was put into operation. The plant adopted the nitrification-denitrification process for the oxidation ditch process first in Japan. At Omihachiman City a batch system for their oxidation ditch was adopted, and started the operation of this plant in 1982. The locations of these plants are shown in Fig. 1, and each treatment capacity is shown in Table 1. The largest plant has a capacity of about 3000 m^3/day.

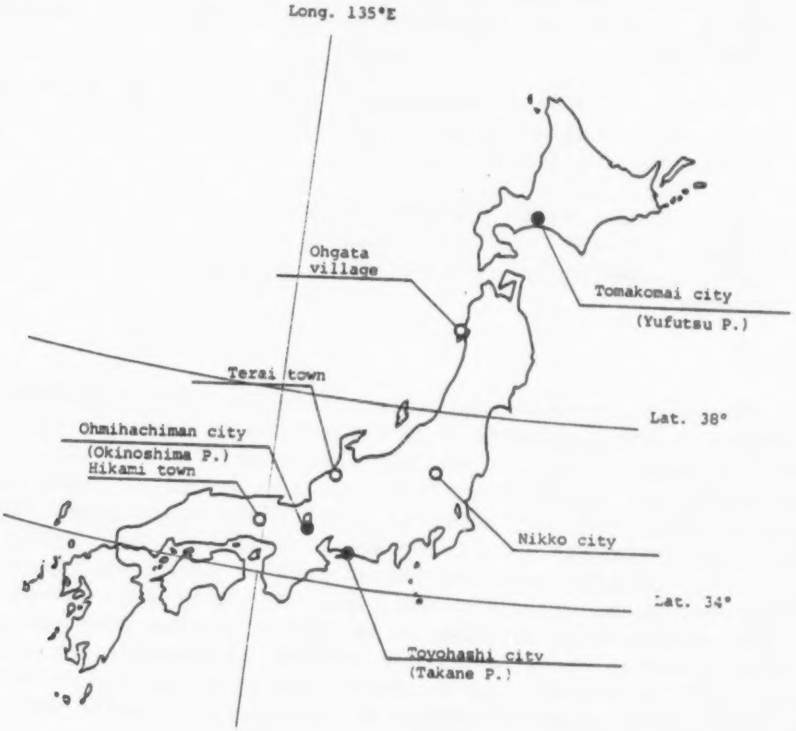

Long. 135°E
Ohgata village
Tomakomai city
(Yufutsu P.)
Terai town
Lat. 38°
Ohmihachiman city
(Okinoshima P.)
Hikami town
Nikko city
Lat. 34°
Toyohashi city
(Takane P.)

Fig. 1 Locations of oxidation ditch plants

Table 1 Oxidation ditches in service

	Treatment capacity (m^3/day)	Mechanical aeration device	Fiscal year, in service
Nikko City	3,084	Horizontal axis type	1966
Ogata Village	1,536	Horizontal axis type	1969
Toyohashi City	650	Axial flow pump	1977
Tomakomai City	2,140	Horizontal axis type	1979
Ohmi Hachiman City	210	Vertical axis type	1982
Terai Cho Town	900	Axial flow pump	1982
Hikami Cho Town	3,560	Vertical axis type	1983

1.3 Prospect for the Oxidation Ditch Process in Japan

As of the end of fiscal 1984, about 35% of total population was served by sewarage system in Japan, while that in ten large cities, i.e. Tokyo, Yokohama, Osaka, Nagoya and so on, was about 80% of their populations. Therefore, it is expected that the central point of constructions of sewer systems is gradually shifting from large cities to local medium-sized or small communities. Table 2 shows the numbers of the wastewater treatment plants, classified by treatment processes, with a treatment capacity of less than 5,000 m^3/day, which were planned for construction during the five years from fiscal 1979 to fiscal 1983.

Table 2 Number of newly projected wastewater treatment plants with a design wastewater capacity of less than 5,000 m^3/day, and changes in the treatment method

(Investigated by the Ministry of Construction)

Fiscal year / Treatment process	1979	1980	1981	1982	1983
Oxidation ditch process	1	5	8	10	9
Extended aeration process	2	3	5	0	0
Conventional activated sludge process	5	4	2	6	8
Others*	5	6	2	4	6
Total	13	18	17	20	19

Note: Others includes the RBC process

According to Table 2, about 20 plants have been planned annually, and about 50% of them were to employ the oxidation ditch process. This may clearly indicate that people place great expectations on the oxidation ditch process for small communities in spite of that only four oxidation ditch plants were in service in 1970s.

Generally, the municipalities planning to build a small sewage system have the following common features:

a. The population is small.
b. The budget is small.
c. Price of land is less expensive than that in large cities.
d. It is difficult to employ skillful personnel.

Therefore, the treatment plants in small communities are required following characteristics.

(1) To treat the wastewater with large fluctuation stably.
(2) To provide easy equipments to maintain.
(3) To meet the water quality standards without skillful personnel.
(4) To be constructed much cheaper than the conventional system.
(5) To be operated and maintained much cheaper than the conventional system.

The oxidation ditch process is designed as a low loading process; the BOD-SS loading ranges from 0.03 to 0.05 kg-BOD/kg-SS/day. The primary settling tank and blower are not usually provided. This system therefore, has a certain potential to meet the requirements of (1), (2), and (3) noted above, which are the requirements for a small wastewater treatment plant. In addition, because its ditch depth is designed to be less than 3 m deep. It is possible to construct the ditch wall surface and floor simply with asphalt or block in order to reduce the construction cost described in (4).

The Japan Sewage Works Agency has been employing the oxidation ditch processes for small wastewater treatment plants of which capacity are less than 5,000 m^3/day (1.32 MGD).

2. SURVEYS ON THE OXIDATION DITCH PROCESS

The oxidation ditch process has become increasingly noticeable as a secondary treatment process for small communities, as its ability to remove nitrogen by providing an aerobic and anoxic portion in a ditch. For this reason, extensive research and investigations have been conducted by various institutions to evaluate the process performance including BOD and suspended solids removal, sludge settlability and sludge production and so on, as well as to evaluate the denitrification characteristics and the aeration devices. Major research projects that have been carried out so far as follows:

(1) Pilot plant studies by the Public Works Research Institute, Ministry of Construction.

(2) Investigations of the actual treatment facilities in service by the Japan Sewage Works Agency.

(3) Performance evaluation of the mechanical aerator for the oxidation ditch process by the Ministry of Construction.

This paper describes the contents and results of these investigations.

2.1 Pilot Plant Study

2.1.1 Outline

The Public Works Research Institute, Ministry of Construction has conducted investigations by using a pilot plant of oxidation ditch for more than three years. The purpose of the investigations were to obtain useful informations for design and maintenance such as treatment characteristics, sludge production and the current distribution in the ditch under various operating conditions. (They have also been doing research on the difference of the treatment characteristics between the continuous flow system and batch system.) The outline of pilot plant facilities is shown in Table 3, and the facility layout is shown in Fig. 2.

The average BOD, soluble BOD and suspended solids in influent were 120, 40, 90 mg/ℓ, respectively, on the average during the investigation.

Table 3 Outline of main facilities of a pilot plant

Ditch capacity	160 - 224 m^3
Hydraulic retention time	48 hours
Ditch width	2 m
Length of circumference	36.2 m
Depth of water	2.0 - 2.8 m
Rotor revolutions	36, 48, 72 rpm
Ditch surface area	80 m^2

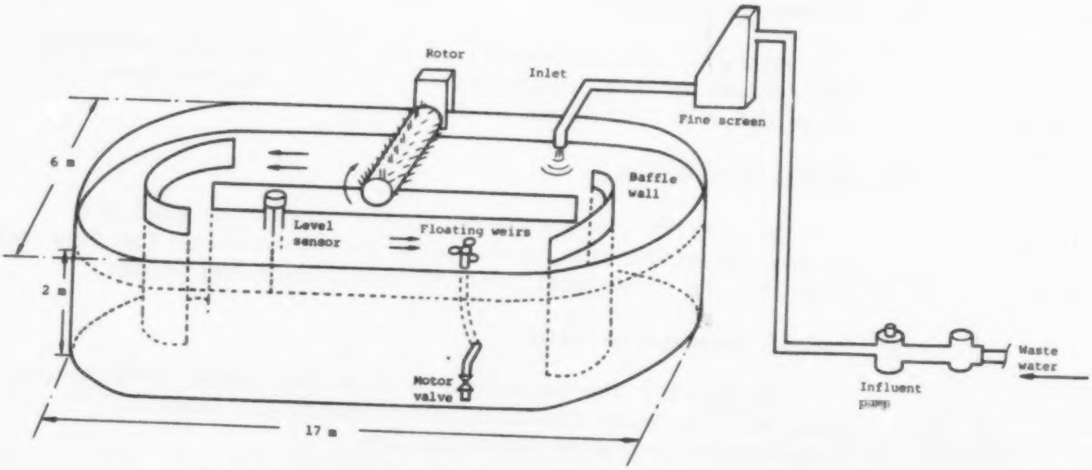


Fig. 2 A sketch for pilot plant (batch system)

2.1.2 Pilot plant study of the continuous flow system

(1) Operating conditions

The experiments were carried out after the activated sludge had been acclimated for about three months. The operating conditions during the experiments were as shown in Table 4. The MLSS concentration during the experiments was 5,200 mg/ℓ on the average and a little higher than that of the design standards.

Table 4 Operational conditions at ditch process pilot plant (continuous system)

Areation time period	24 hours
BOD - SS loadings	0.028 kg-BOD/kg MLSS/day
BOD volumetric loadings	0.147 kg-BOD/m^3/day
MLSS	Average 5,200 mg/ℓ (2330 - 8620)
SRT	56 days or no drawing
Return sludge ratio	100%, partially 150%
Retention time in final settling tank	3.5 hours
Overflow rate of final settling tank	13.8 m^3/m^2/day

(2) Contents of survey

The major items of the survey were as follows:

① Process performance

To study the process stability of wastewater regarding the removal of BOD, suspended solids and nitrogen under the large daily fluctuations of loadings.

② Sludge production

To clarify the amount of sludge production when the sludge retention time is fixed as a certain value (56 days are used for the experiment) during both summer and winter.

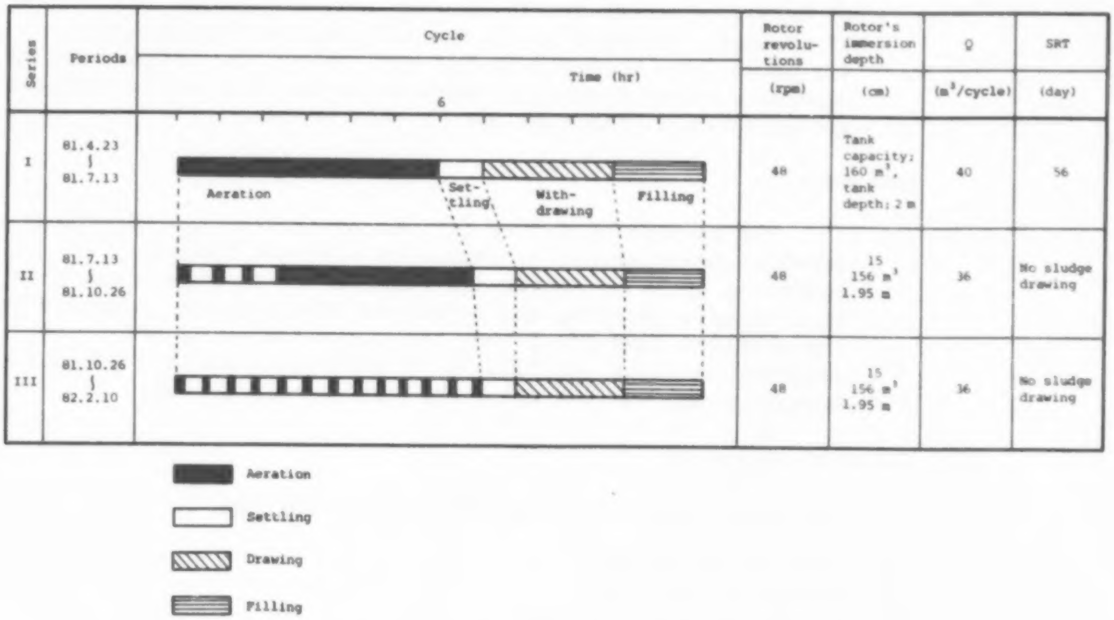
③ Oxygen consumption rate

To clarify the relations between the oxygen consumption rate and the daily fluctuations of loadings, water temperature, and MLSS concentration.

2.1.3 Experiment of batch system

(1) Operating conditions

As for batch system, the experiments were carried out under the three different operating conditions as shown in Fig. 3.



Series	Periods	Cycle	Rotor revolutions	Rotor's immersion depth	Q	SRT
		Time (hr)	(rpm)	(cm)	(m^3/cycle)	(day)
I	81.4.23 ∫ 81.7.13		48	Tank capacity; 160 m^3, tank depth; 2 m	40	56
II	81.7.13 ∫ 81.10.26		48	15 156 m^3 1.95 m	36	No sludge drawing
III	81.10.26 ∫ 82.2.10		48	15 156 m^3 1.95 m	36	No sludge drawing

Fig. 3 Operating conditions of the batch process

These three conditions were:

(Series I)

The purpose of series I was to clarify the effects of a complete nitrification. An operating cycle of series I consists of the aeration period for 6 hours, the settling period for 1 hour, and withdrawing of supernatant and filling of wastewater for 5 hours. The sludge retention time on this series was set as being 56 days.

(Series II)

The purpose of series II was to clarify the influences given by an extremely long sludge retention time to the process performance and the amount of sludge production, and the effects of aerobic-and-anaerobic operation on the process performance.

An operating cycle of series II consists of three cycles of aeration period for 15 minutes and idle period for 30 minutes, followed by the second aeration period for 4.5 hours, the settling period for 1 hour, and withdrawing of supernatant and feeding of wastewater for 4.25 hours. Excess sludge was not withdrawn during this series.

(Series III)

The purpose of series III was to clarify the effects of aerobic-and-anaerobic operation on nitrogen removal. An operating cycle of series III consists of 10 times repetitions of 15 minute aeration and 30 minute idle periods, followed by the settling time for 1 hour and the withdrawing of supernatant water and filling of wastewater for 4.5 hours. Excess sludge was not withdrawn during this series.

(2) Contents of survey

The major items of the survey were as follows:

① Process performance
② Sludge production
③ Oxygen consumption rate
④ Velocity distribution in the ditch

2.2 On-site Survey

2.2.1 Outline

The Japan Sewage Works Agency made an investigation into the problems of oxidation ditch in service in connection with the removal of BOD, suspended solids and nitrogen, and maintenance. The Agency has been making a technical evaluation of the oxidation ditch process based on the results of the investigation. The periods of investigation were three years from April 1982 to March 1985, and generally, the period for one treatment plant was one year.

2.2.2 Outline and the operating conditions of the surveyed treatmenc plants

(1) Yufutsu wastewater treatment plant

The Yufutsu wastewater treatment plant is connected to a separate sewer system and its influent is basically a domestic wastewater. The outline of the facilities is shown in Table 5, the treatment flow diagram is shown in Fig. 4, and the operating conditions are shown in Table 6.

Table 5 Outline of major facilities of the Yufutsu plant

System / Facility	First system	Second system
Main pump	Submerged motor pump ø100 x 2 units, Q = 1.6 m^3/min per unit Submerged motor pump ø450 x 2 units, Q = 4.0 m^3/min per unit	
Oxidation ditch	6.0 m (width) x 154 m (length) x 1.2 m (water-depth) = 1,100 m^3 (capacity) Aeration time; 24.7 hours* (11.5)** BOD-SS loading; 0.085 kg/SSkg/day*** BOD volumetric loading; 0.20 kg/m^3/day*	6.0 m (width) x 154 m (length) x 1.4 m (water-depth) = 1.283 m^3 (capacity) Aeration time; 28.8 hours* (13.4)**
Final settling tank	4.0 m (width) x 41.0 (length) x 1.2 m (depth) x 2 (ponds) Capacity; 392 m^3, Length of weir; 36.9 m Detention time; 4.4 hours* (2.0)** Overflow rate; 6.5 m^3/m^2/day (14.0)** Discharge per unit length; 29.0 m^3/m/day (62.4)**	4.0 m (width) x 41.0 m (length) x 1.4 m (water-depth) x 2 (ponds) Capacity; 456 m^3, Length of weir; 36.9 m Detention time; 5.1 hours* (2.4)** Overflow rate; 6.5 m^3/m^2/day* (14.0)** Discharge per unit length; 29.0 m^3/m/day* (62.4)**
Main rotor I	Cage type; ø820 mm, L = 5720 mm Immersion depth; 160 mm, 10 - 80 rpm, 37 kW	Cage type; ø820 mm, L = 5720 mm Immersion depth; 160 mm, 30, 45, 60 rpm, 22 kW
Main rotor II	Bursh type; ø1000 mm, L = 4500 m Immersion depth; 175 mm, 40 - 80 rpm	
Sub-rotor	Cage type; ø820 mm, L = 3720 mm, Immersion depth; 175 mm, 60 rpm, 11 kW	
Thickener	ø3.8 m x (water-depth) 2.0 m x 2 (units)	ø3.8 m x 2.0 m (water-depth) x 2 (units)

Notes: * A value corresponding to the schematic daily maximum flow, 1,070 m^3/day.

** The numerical value in the parentheses expresses a value corresponding to the capacity of main pump (1.6 x 24 x 60 = 2,304 m^3/day).

*** The measured value of BOD-SS loadings in fiscal 1981 and 1982 were 0.01 to 0.09 kg-BOD/kg-SS/day.

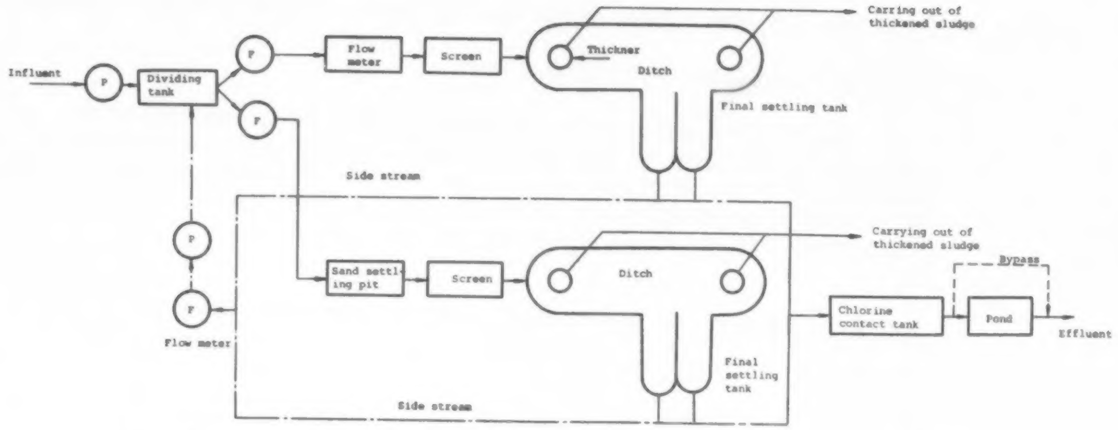

Influent
P
Dividing tank
P
Flow meter
Screen
Thickner
Ditch
Carring out of thickened sludge
Final settling tank
P
Side stream
P
F
Flow meter
Sand settling pit
Screen
Ditch
Carrying out of thickened sludge
Bypass
Chlorine contact tank
Pond
Effluent
Final settling tank
Side stream

Fig. 4 Flow diagram of the Yufutsu wastewater treatment plant

Table 6 Operation condition (Yufutsu treatment plant)

Research period	Series number	Aeration unit operation condition		Oxygen supply rate kg-O_2/day	Average inflow rate (minimum - maximum) m^3/day	BOD-SS loadings kg-BOD/kg-SS/day	Water temperature in ditch °C	MLSS concentration mg/ℓ
		Main rotor I (number of rotations)	Main rotor II (number of rotations)					
1982.9.1 - 10.29	1	Continuous (45)	-	-	-	-	-	-
	2	Continuous (60)	-	140	609 (467 - 892)	0.026 - 0.045	14 - 19	2810 - 3610
1982.10.30 - 12.24	1	Intermittent: 10 hrs/day (60)	Continuous (45)	230	614 (548 - 686)	0.018 - 0.037	9.0 - 12.8	3900 - 5320
	2	-	Continuous (60)	170	594 (520 - 748)	0.017 - 0.034	9.0 - 12.7	3900 - 4500
1982.12.25 - 1983.2.16	1	Intermittent: 10 hrs/day (60)	Continuous (45)	230	518 (471 - 599)	0.016 - 0.039	4.2 - 7.5	3800 - 5400
	2	Continuous (60)	Continuous (60)	310	519 (477 - 570)	0.013 - 0.036	4.2 - 6.9	3700 - 4900
1983.5.26 - 7.14	1	-	Continuous (60)	210	706 (516 - 1025)	0.023 - 0.073	12.1 - 15.8	3170 - 4010
	2	Intermittent: 10 hrs/day (60)	Continuous (60)	240	661 (416 - 1183)	0.018 - 0.068	12.0 - 15.3	3380 - 4520

Notes: 1. The main rotor I stops during operation of the final settling tank mechanical aeration unit (sub-rotor).
2. The number of main rotor rotations is shown by rpm.
3. The oxygen supply rate refers to the estimate (calculated value) by KLa_{20} measured in the mixed liquor.

(2) Takane wastewater treatment plant

The Takane wastewater treatment plant is connected to a separate sewer system in a rural area. Because its influent consists of domestic wastewater and livestock waste, the wastewater is characterized by its greater concentration of BOD and nitrogen compared with general wastewater. The outline of the facilities is shown in Table 7, the treatment flow diagram is shown in Fig. 5, and the operating conditions are shown in Table 8.

Table 7 Outline of major facilities of the Takane plant

Facility	Structure and specification
Main pump	Submerged pump 0.5 m^3/min. x 11 m x 4.5 kW x 1 (unit)
Imhoff tank	6.0 m (width) x 6.0 m (length) x 7.5 m (Depth) Capacity ; 238 m^3 Water area; 36 m^2
Oxidation ditch	6.0 m (width) x 200 m (length) x 1.1 m (water-depth) Capacity ; 1,320 m^3 Aeration time ; 48 hours* BOD-SS loadings ; 0.05 kg-BOD/kg-SS/day* BOD volumetric loadings; 0.08 kg-BOD/m^3/day*
Final settling tank	6.0 m (width) x 6.0 m (length) x 3.0 m (water-depth) Capacity ; 100 m^3 Water area ; 36 m^3 Overflow rate ; 18 m^3/m^2* Detention time; 3.7 hours*
Mechanical aerator	Axial flow pump ø1200 x 11 kW x 2 (units) Roots type blower; 4.0 m^3/min. x 0.22 kg/cm^2 x 3.7 kW x 1 (unit)
Sludge drying beds	Total sand bed area; 245 m^2

Note: Those with the asterisk mark are the values at design.

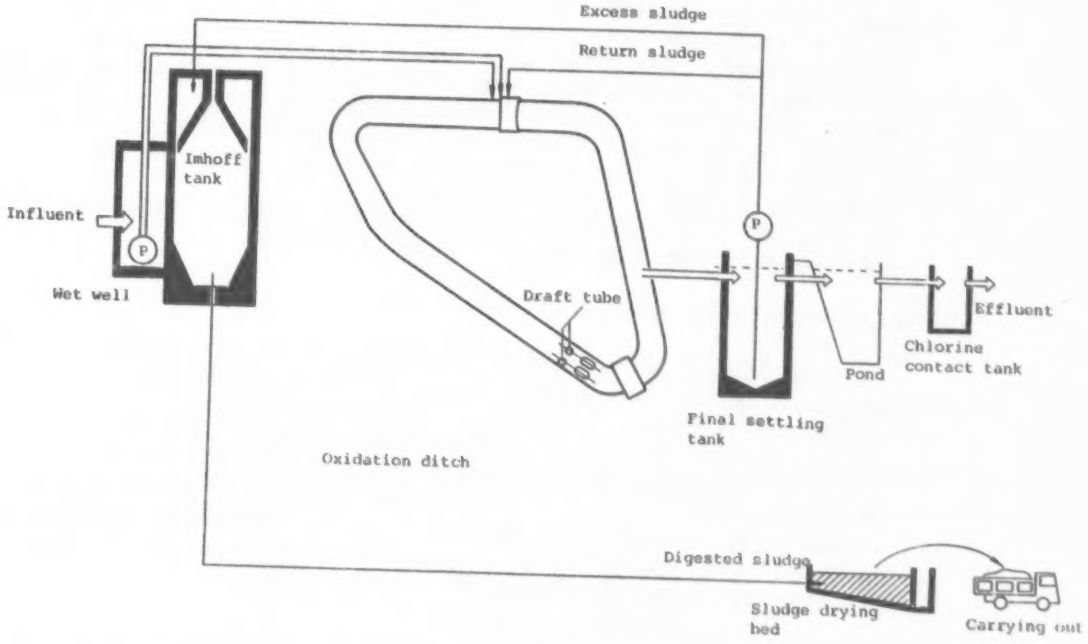

Excess sludge
Return sludge
Imhoff tank
Influent
P
Wet well
Draft tube
P
Effluent
Chlorine contact tank
Pond
Final settling tank
Oxidation ditch
Digested sludge
Sludge drying bed
Carrying out

Fig. 5 Flow diagram of the Takane wastewater treatment plant

Table 8 Operation condition (Takane treatment plant)

Research period	Air-flow rate m^3/min.	Average inflow rate (minimum - maximum) m^3/day	BOD-SS loadings kg-BOD/ kg-SS/day	Water temperature in ditch °C	MLSS concentration mg/ℓ
1983.7.25 - 8.19	1.9	318 (274 - 386)	0.010 - 0.022	27.2 - 31.8	2820 - 3540
1983.10.17 - 11.25	0.8	309 (264 - 399)	0.010 - 0.018	12.5 - 21.2	3480 - 3900
1983.12.2 - 12.22	0.7	270 (251 - 280)	0.011 - 0.016	7.8 - 10.4	3590 - 4160
1984.2.27 - 3.23	1.6	291 (249 - 424)	0.015 - 0.019	6.5 - 9.0	3150 - 3780

To clarify the effects of the increase or decrease of the aerobic zone in the ditch on the nitrogen removal, this treatment plant is operated by changing air flow rate from blowers into two stages. Aerator mounted in Takane plant consists of an axial flow pump for mixing and a blower for oxygen supply into mixed liquor. Therefore, it is possible to control oxygen supply easily. One operating mode (called the full nitrification mode) is intended to facilitate the complete nitrification as much as possible with the air flow rate from 1.6 to 1.9 m^3/min. (7.6 to 9.0 m^3 per 1 m^3 of wastewater). The other is an operating mode (called the nitrification control mode) that controls the nitrification to the extent whereby the effluent quality (mainly T-BOD) is not adversely affected, with the air flow rate from 0.6 to 1.0 m^3/min. (2.9 - 4.8 m^3 per 1 m^3 of wastewater.) The DO distribution in the ditch in each mode is shown in Fig. 6.

The effluent qualities of the Yufutsu plant and the Takane plant were analyzed by taking composite samples.

(3) Okinoshima wastewater treatment plant

The Okinoshima wastewater treatment plant was constructed on an island in the Lake Biwa and its wastewater is mainly general domestic wastewater. To prevent the lake water from eutrophication, this plant was planned to employ a nitrification and denitrification system with a batch operation mode at the time of design. The outline of facilities is shown in Table 9, and the treatment flow diagram is shown in Fig. 7. The plant is presently operated with two cycles per day and its time sequence is shown in Fig. 8, and operating conditions in Table 10.

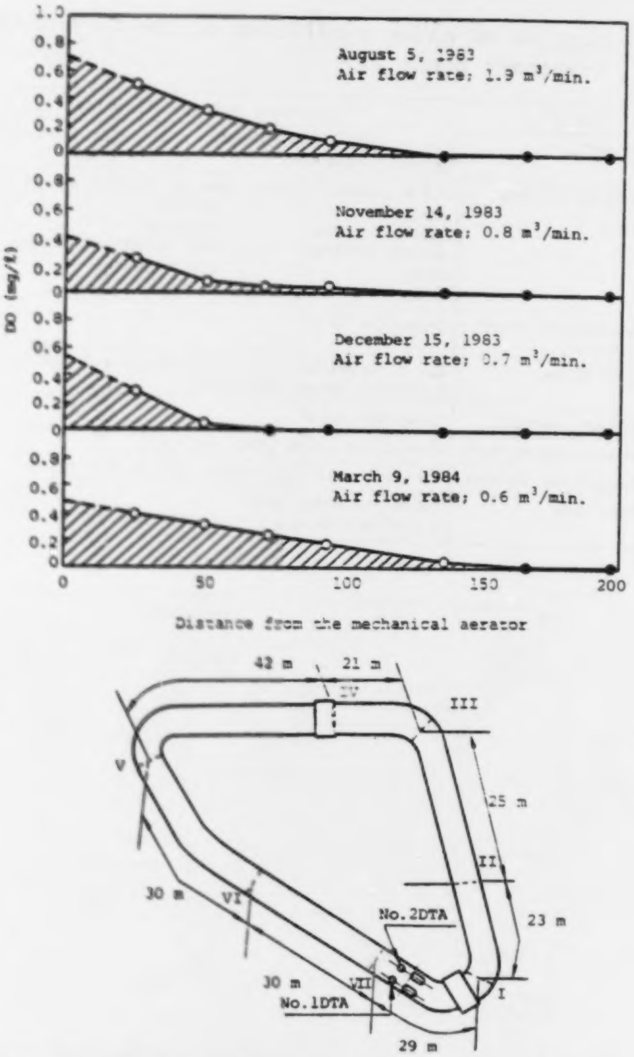


Fig. 6 DO distribution in each operating mode

Table 9 Outline of major facilities of the Okinoshima sewage treatment plant

Facility	Structure and specification
Wet well	8 m (width) x 9 m (length) x 1.5 m (water-depth) Capacity; 108 m^3
Oxidation ditch	2.5 m (width) x 68 m (length) x 2.5 m (water-depth) Capacity ; 425 m^3 Detention time ; 48 hours BOD-SS loadings ; 0.05 kg/-BOD/kg-SS/day MLSS ; 3000 mg/ℓ Sludge detention time; 30 days
Supernatant water reservoir	Capacity; 50 m^3
Sand filter	1.8 m (diameter) x 4.5 m (height) x 2 (units) Filtering speed; 200 m/day
Sludge reservoir	Capacity; 82 m^3
Sludge drying bed	Sand floor total area; 64.3 m^2
Mechanical aerator	Vertical axis type rotor; 7.5 kW x 1 (unit)
Main pump	Submerged pump; ø150 1.8 m^3/m x 10 m x 7.5 kW x 2 (units)

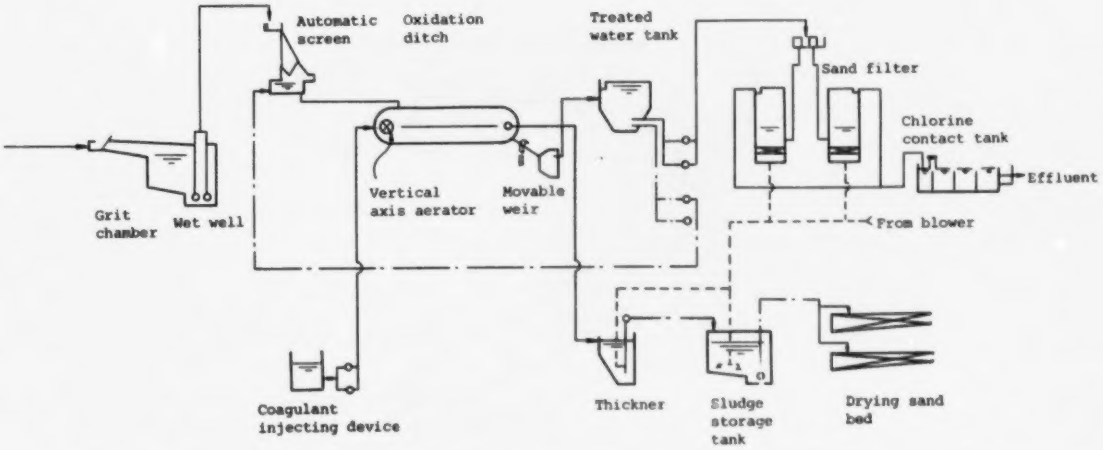
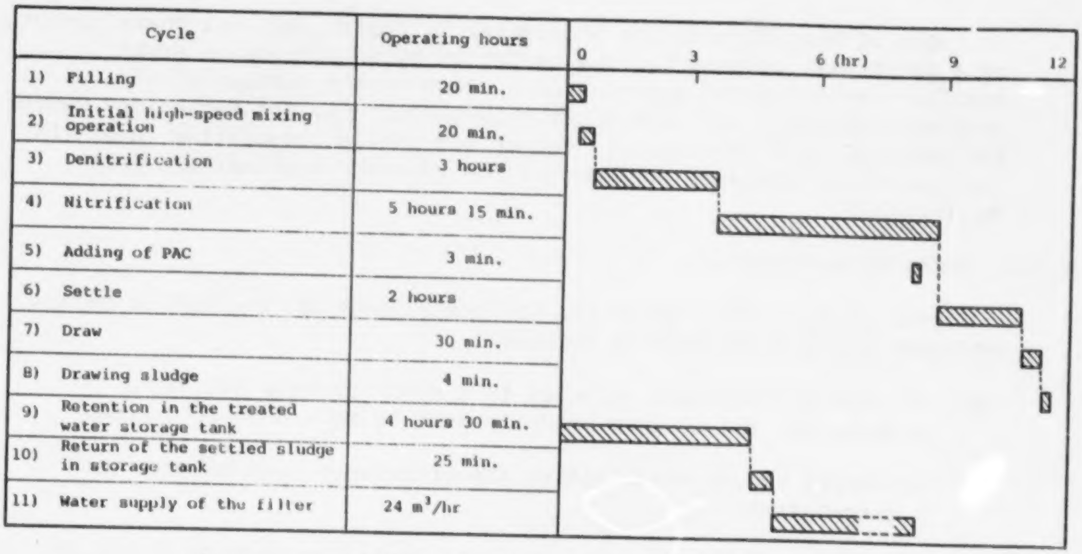

Automatic screen
Oxidation ditch
Treated water tank
Sand filter
Chlorine contact tank
Effluent
Grit chamber
Wet well
Vertical axis aerator
Movable weir
From blower
Coagulant injecting device
Thickner
Sludge storage tank
Drying sand bed

Fig. 7 Flow diagram of the Okinoshima wastewater treatment plant



Cycle	Operating hours
1) Filling	20 min.
2) Initial high-speed mixing operation	20 min.
3) Denitrification	3 hours
4) Nitrification	5 hours 15 min.
5) Adding of PAC	3 min.
6) Settle	2 hours
7) Draw	30 min.
8) Drawing sludge	4 min.
9) Retention in the treated water storage tank	4 hours 30 min.
10) Return of the settled sludge in storage tank	25 min.
11) Water supply of the filter	24 m^3/hr

Fig. 8 Operating schedule of the Okinoshima plant

Table 10 Operation condition (Okinoshima sewage treatment center)

Research period	Number of aeration unit rotations rpm	Average inflow rate (minimum - maximum) m^3/day	BOD-SS loadings kg-BOD/kg-SS/day	Water temperature in ditch °C	MLSS concentration mg/ℓ
1984.7.1 - 9.30	21 (Denitrification) 39 (Nitrification)	94.3 (85.1 - 102.7)	0.012 - 0.021	22.8 - 29.7	3500 - 4960
1984.10.1 - 12.31	21 (Denitrification) 39 (Nitrification)	89.9 (73.6 - 97.1)	0.012 - 0.023	10.3 - 24.5	3450 - 5200
1985.1.1 - 3.31	21 (Denitrification) 39 (Nitrification)	94.4 (83.9 - 137.7)	0.011 - 0.018	7.7 - 11.3	4100 - 5210

2.3 Assessment of the Mechanical Aerators

2.3.1 Construction technology assessment system

This system is to assess the functions and performances of new technology developed by manufactures. First, the Minister of Construction announces the technical subjects to be assessed in an official publication, inviting the manufacture to submit their proposal. The Minister of Construction issues a certificate of its performance after the assessment by the Technology Assessment Committee organized by the Ministry of Construction. This system contribute to promote the application of new technology as well as technology development.

One of the subjects for assessment in fiscal 1981 was "Development of a mechanical aerator for the oxidation ditch process". Eight manufactures submitted applications. Because the submission of applications continued even after this assessment, the Technology Center for National Land Development, one of the public corporation under the control of the Ministry of Construction assessed applications from manufacture.

2.3.2 Goals of development

The predetermined goals for the development of the aerator on the official publication were as follows:

(1) To assure sufficient velocity in a ditch to keep solids in suspension; the minimum velocity must be more than 10 cm/sec.

(2) To supply oxygen efficiently; the efficiency must be more than 1.8 kg-O_2/kWh.

(3) To provide a sufficient mixing; the deviation of MLSS within any cross section must be less than 10%.

(4) To assure easy maintenance.

(5) To be in structure that measures can be easily taken for protection of surrounding environment.

2.3.3 Results of assessment

The 13 applications, of which; 7 in the horizontal axis type, 3 in the vertical axis type and 3 in the others, were submitted in fiscal 1981 to 1982. All aerators were assessed as being capable of satisfying the goals of development.

More detail information is in the proceeding of the 9th US/Japan Conference on Sewage Treatment Technology entitled "Construction technology assessment system and the development of a mechnaical aerators for oxidation ditch process".

3. PROCESS PERFORMANCE

3.1 BOD Removal

The sludge retention time of this process is long so that the nitrification is progressed easily. Therefore, effluent BOD consists of the carbonaceous oxygen demand and oxygen demand by nitrification. As the carbonaceous oxygen demand was measured as nitrification inhibited BOD by addition of allylthiourea (called ATU-BOD), BOD by nitrification (called N-BOD) was calculated by subtracting ATU-BOD from ordinary BOD (called T-BOD).

Fig. 9 shows cumulative frequency distribution curves of effluent BOD. The data shown in the figure are of T-BOD of the three plants selected for the field survey, and soluble BOD (called S-BOD) and ATU-BOD of the Yufutsu plant. According to Fig. 9, in the continuous flow system effluent T-BOD occasionally exceeded 20 mg/ℓ (effluent T-BOD with the oxidation ditch process is regulated to be less than 20 mg/ℓ by the effluent standards in Japan), whereas effluent ATU-BOD was generally less than 10 mg/ℓ during investigations. Thus, it is important to decrease N-BOD in order to meet the effluent standards.

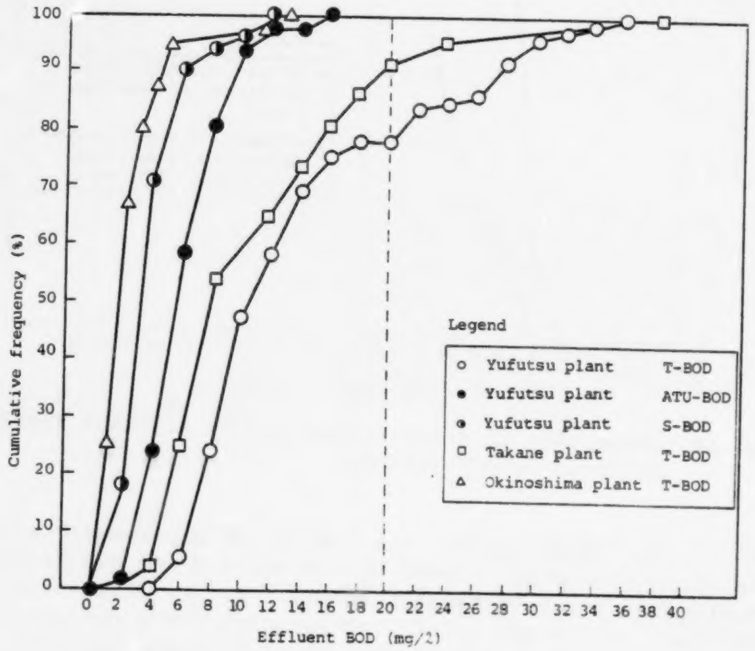


Fig. 9 Cumulative frequency distribution curve of T-BOD, ATU-BOD and S-BOD of the effluent

Fig. 10 shows the cumulative frequency distributions of T-BOD of effluent samples with Kjeldahl nitrogen more than 2 mg/ℓ and less than 2 mg/ℓ at the Yufutsu plant.

It is clear from the figure that nitrification greatly affects effluent T-BOD. That is, the value of T-BOD are always smaller than 20 mg/ℓ in case of the well nitrified effluent. On the other hand, the frequency exceeding the effluent T-BOD 20 mg/ℓ was about 50% in case of the partial nitrification. Therefore, one solution would be to accomplish the nitrification completely in order to meet the effluent standards.

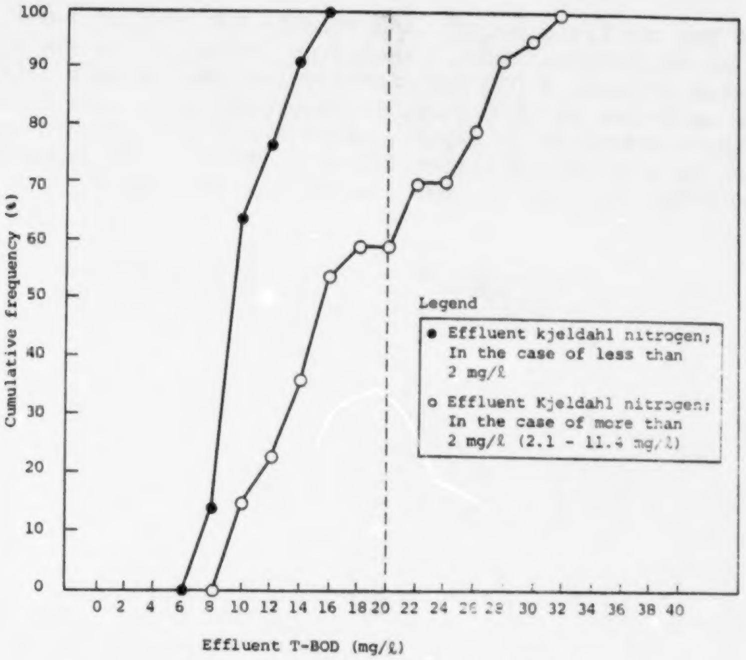


Fig. 10 Comparison of T-BOD classified by effluent Kjeldahl nitrogen concentration

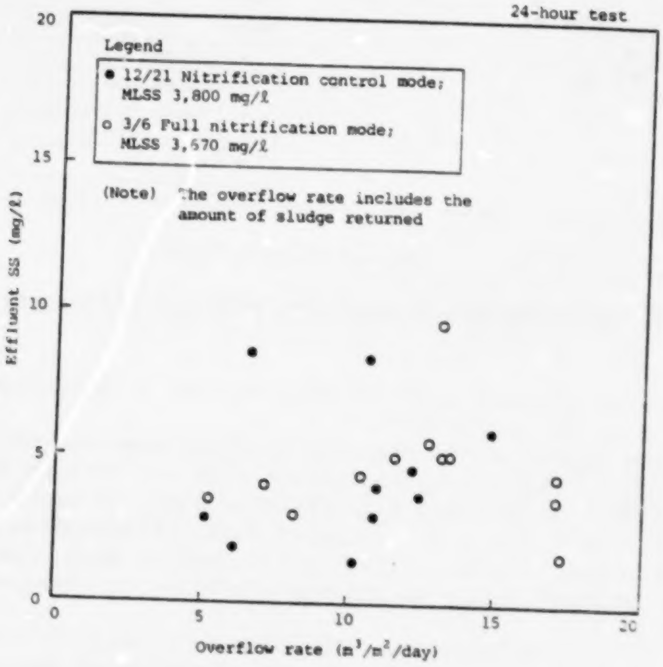
As for the batch system, the results of experiments at pilot plants and the Okinoshima plant showed that T-BOD did not exceed 20 mg/ℓ, and this system can treat wastewater stably. At the Okinoshima plant, the nitrification progressed sufficiently except for a certain period in winter, and the effluent concentration of Kjeldahl nitrogen was 1.6 mg/ℓ on the average. About 90% of the data were less than 1.0 mg/ℓ.

According to the 24-hour investigations at the Takane plant, effluent S-BOD with respect to the changes of the effluent quality in time was in the range of about 2 mg/ℓ to 7 mg/ℓ, and influent BOD was treated stably even when the hourly maximum flow was about 2.2 times of the design capacity.

3.2 Suspended Solid Removal

As for effluent suspended solids at each treatment plant, the maximum and the minimum were 26, 1 mg/ℓ, respectively. Therefore, if the oxidation ditch process is operated under BOD loadings from 0.01 - 0.07 kg-BOD/kg-SS/day, stable SS removal could be achieved.

Fig. 11 shows the relationship between the overflow rate of the final settling tank and the effluent suspended solid at the Takane plant. The overflow rate in this case refers to a value based on the total flow of wastewater and return sludge. Fig. 11 shows that the overflow rate remained within the range from 5 to 20 m^3/m^2/day, and the effluent suspended solids never exceeded 10 mg/ℓ. Also, the experiments (experiments on the continuous treatment system) by the Public Works Research Institute showed the removal of suspended solids was stable in the range from 6.9 to 26.1 m^3/m^2/day of the overflow rate.



(Note) The overflow rate includes the amount of sludge returned

Fig. 11 Relationship between the overflow rate and effluent suspended solids in the final settling tank

In the oxidation ditch process, the MLSS concentration is maintained at a high concentration of 3,000 to 5,000 mg/ℓ. For this reason, zone settling of activated sludge usually occur in the final settling tank, so that fine flocs are held in sludge blanket. These results show that removal of suspended solids is excellent if the suitable overflow rate is set for the final settling tank and withdrawing of activated sludge is carried out appropriately. The batch system especially exhibits this characteristics because of the zone settling.

3.3 pH

Fig. 12 shows the relationship between the alkalinity and pH in effluent at three plants.

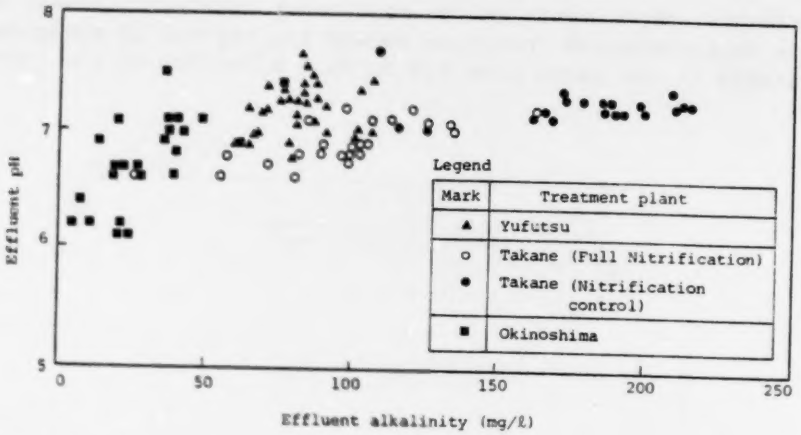


Fig. 12 Relationship between the effluent alkalinity and pH

pH never declined significantly because of sufficient recovery of alkalinity by denitrification.

At the Takane plant, distinctive difference was noticed between effluent pH in the full nitrification mode and that in the nitrification control mode. In the full nitrification mode, pH was a little low in the range from 7.1 to 6.6 because of full nitrification and partial denitrification. In the nitrification control mode, however, due to complete denitrification of nitrate produced, pH remained in the range from 7.0 to 7.3.

At the Okinoshima plant, the effluent alkalinity was considerably low compared to other two plants, and pH sometimes declined to 6.2 as shown in Fig. 12. However, at this plant, the alkalinity in the mixed liquor changes with aeration time (nitrification and denitrification process) as shown in Fig. 13. The average pH in the mixed liquor during aeration is much higher compared with effluent pH.

Figs. 12 and 13 show that if pH decreases as nitrification progresses, the decrease in pH can be prevented by incorporating denitrification reaction in a ditch to recover alkalinity as required.

3.4 Nitrogen Removal

3.4.1 Nitrification

The relationship between influent Kjeldahl nitrogen into the ditch and that of effluent of each treatment plant is shown in Fig. 14.

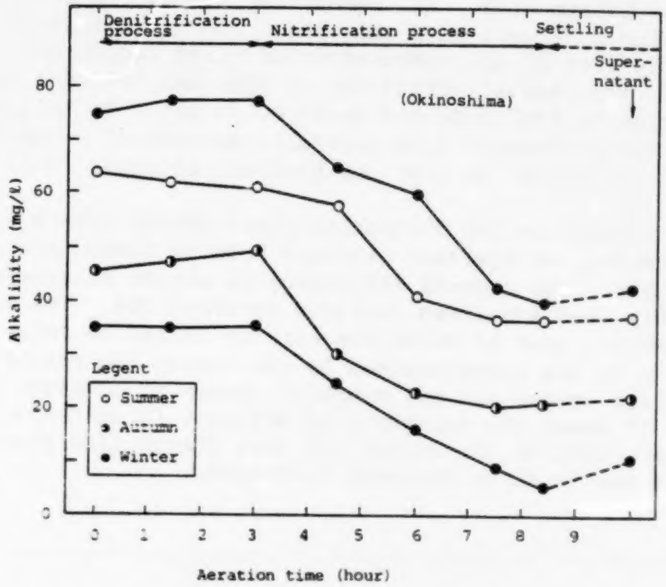


Fig. 13 Changes in the alkalinity of mixed liquor

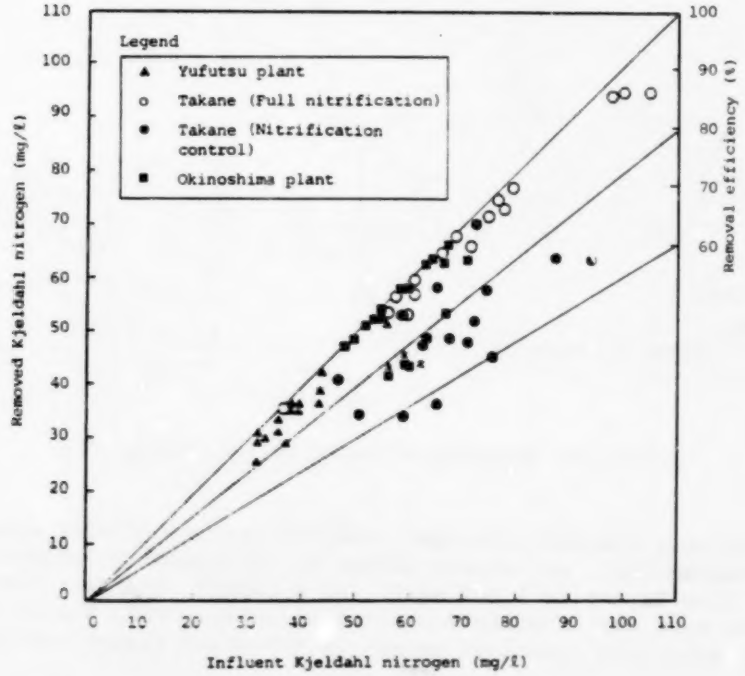


Fig. 14 Characteristics of the nitrification

At the Takane plant, the removal of Kjeldahl nitrogen in the full nitrification mode were more than 90%. The nitrification occurred sufficiently even if the temperature of mixed liquor was 6°C. On the other hand, the removal efficiency of Kjeldahl nitrogen in the nitrification control mode was about 60 to 90%. In this mode, nitrification progressed just partially because of a smaller aerobic portion in the ditch, so that the Kjeldahl nitrogen remained in the effluent.

Observations on the Okinoshima plant (Batch type system) showed that the removal of Kjeldahl nitrogen both in summer and in autumn was more than 95%. The removal efficiency in winter fluctuated considerably, and sometimes declined to about 80%. The asterisk mark in Fig. 14 shows a case in which the aerator failed to raise the revolutions to the predetermined 39 rpm during the nitrification cycle and dissolved oxygen was not supplied enough to complete nitrification.

Fig. 15 shows the variation of effluent soluble nitrogen from Nov. 1981 to Feb. 1982 of the series III test (Batch type process) carried out by the Public Works Research Institute.

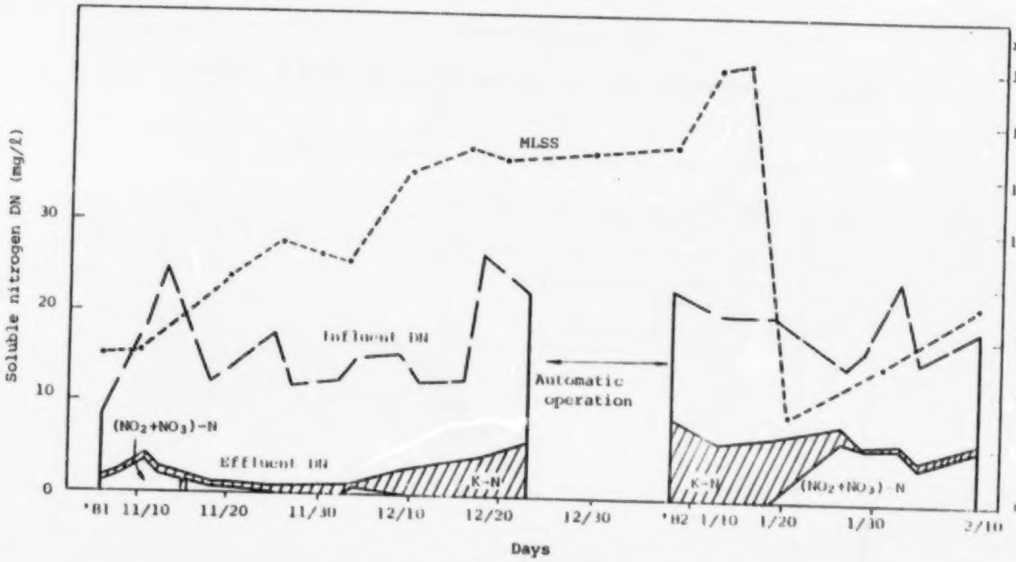


Fig. 15 Removal nitrogen in the series III

Soluble Kjeldahl nitrogen remained in the effluent after the middle of December 1981, and excess sludge was withdrawn to reduce MLSS in the ditch from 16,000 to 3,000 mg/ℓ. As a result, Kjeldahl nitrogen was seen to have converted into nitrate nitorgen. This may be attributable to the increased dissolved oxygen in the mixed liquor caused by lowering MLSS.

All these observations suggest that the amount of air supplied is a control factor to determine the degree of nitrification.

The relationship between the water temperature and the nitrification rate of Kjeldahl nitrogen in the ditch is as shown in Fig. 16.

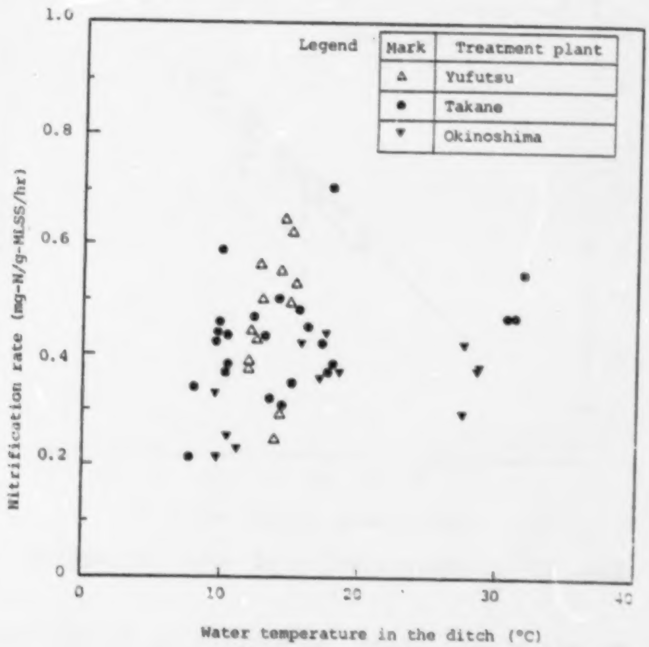


Fig. 16 Nitrification rate in the aerobic zone

The nitrification rate at the Yufutsu and Takane plants was calculated by using the amount of removed Kjeldahl nitrogen and aerobic ditch volume that is effective for nitrification. In the case of the Okinoshima plant, the nitrification rate was calculated as the removal rate of Kjeldahl nitrogen in the ditch assuming a zero order reaction. The nitrification rate was from 0.2 to 0.7 mg-N/g-MLSS/hr and are considerably lower compared with that of nitrified liquor recycled biological nitrification denitrification process (NLRB process), of which rate was 1.6 mg-N/g-MLSS/hr (20°C). It is considered that this low nitrification rate is caused by the low DO distribution in the ditch.

3.4.2 Denitrification

The relationship between the removed Kjeldahl nitrogen concentration and the removed nitrogen concentration of each sewage treatment plant is shown in Fig. 17.

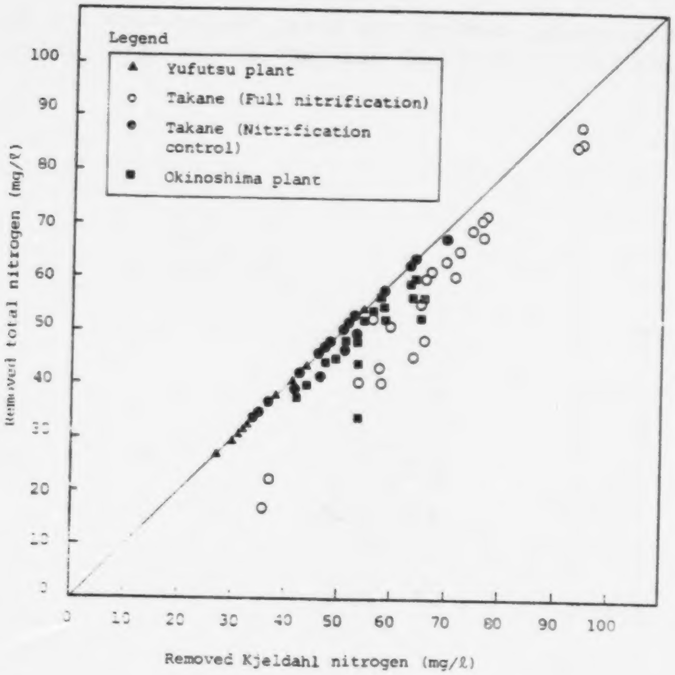


Fig. 17 Characteristics of denitrification

At the Takane plant, almost no nitrite or nitrate remains in the effluent in the nitrification control mode. In the full nitrification mode, however, nitrite and nitrate remain in the effluent owing to nitrification and the total nitrogen removal was smaller than the Kjeldahl nitrogen removal.

At the Okinoshima plant which is operated in a batch mode, nitrate and nitrite nitrogen also remained in the effluent throughout the observations, because the settling and withdrawal cycles follows right after the nitrification cycle in the time sequence. Therefore, the amount of removed total nitrogen was smaller than that of removed Kjeldahl nitrogen.

The relationship between the temperature and denitrification rate of the mixed liquor is shown in Fig. 18.

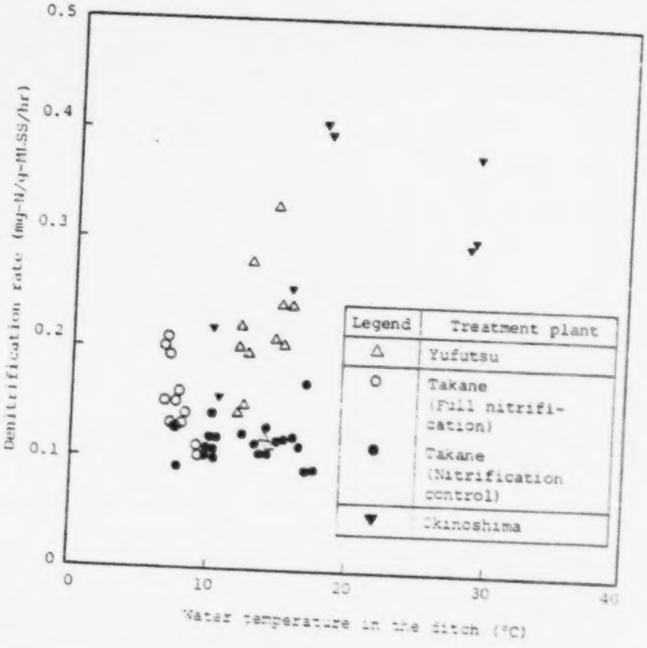


Fig. 18 An average denitrification rate in the ditch

In the case of the Yufutsu plant and the Takane plant, the denitrification rate was calculated by using the difference between influent Kjeldahl nitrogen and the effluent total nitrogen, and ditch volume in which DO is as low as about 0.5 mg/ℓ. In the case of the Okinoshima plant, the denitrification rate was calculated as a removal rate of nitrate nitrogen. The denitrification rate of the oxidation ditch process ranged from 0.1 to 0.3 mg-N/g-MLSS/hr, which is fairly slower than that of the NLRB process, the rate of 0.6 mg-N/g-MLSS/hr at 10°C and 1.0 at 20°C. However, the denitrification rate of the NLRB process could be as low as 0.35 mg-N/g-MLSS/hr when the last portion of the nitrification tank with low soluble BOD (1 - 3 mg/ℓ) was used for denitrification. Therefore, limitation of available BOD as a hydrogen donor is considered to be the cause of the low denitrification rate in the oxidation ditch process.

According to Fig. 18, the denitrification rate in the nitrification control mode is smaller compared with that in the full nitrification mode. This may be caused by the difference of nitrate production between two modes.

3.4.3 Alkalinity balance

Fig. 19 shows the relationship between the calculated alkalinity consumption and the measured alkalinity consumption. These values are expressed as following formulas;

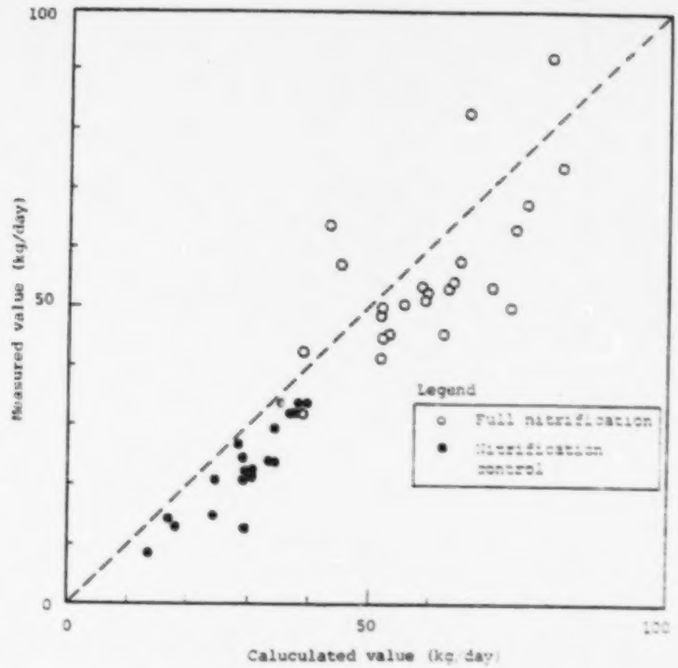


Fig. 19 Alkalinity balance

$$A_M = (C_{Alin} - C_{Aleff}) \cdot Q$$
$$A_C = \{(C_{Ain} - C_{Aeff}) \cdot 7.14 - (C_{Ain} - C_{Neff}) \cdot 3.57\} \cdot Q$$

A_M : Measured alkalinity consumption (kg/day)
A_C : Calculated alkalinity consumption (kg/day)
C_{Alin} : Alkalinity in influent (mg/ℓ)
C_{Aleff}: Alkalinity in effluent (mg/ℓ)
C_{Ain} : NH_4^+ - N in influent (mg/ℓ)
C_{Aeff} : NH_4^+ - N in effluent (mg/ℓ)
C_{Neff} : NO_2^- - N and NO_3^- - N in effluent (mg/ℓ)

Fig. 19 shows that the measured values are almost equal to the calculated values.

Alkalinity balance obtained by other experiments was also the same as above.

3.4.4 Nitrogen removal

The results of experiments at the pilot plants (refer to Fig. 3) are sown in Table 11.

Table 11 Results on the batch system at the pilot plants

	Periods		BOD_{ATU} (mg/ℓ)	COD_{Mn} (mg/ℓ)	SS (mg/ℓ)	VSS (mg/ℓ)	TN (mg/ℓ)	DN (mg/ℓ)	KN (mg/ℓ)	DKN (mg/ℓ)	NH_4-N (mg/ℓ)	NO_2-N (mg/ℓ)	NO_3-N (mg/ℓ)	Water temperature (°C)	pH (-)	Transparency (cm)
Influent	I	Average Range	121 73.4 - 160	72.3 52.2 - [illegible]	122 56.0 - 246	75.9 31.0 - 170	19.1 12.4 - 28.7	14.0 9.51 - 18.7	18.2 11.7 - 28.2	13.2 8.74 - 18.2	9.60 5.23 - 12.1	0.11 0.06 - 0.30	0.71 0.02 - 3.01	19.4 16.5 - 23.3	7.36 7.12 - 7.50	5.0 1.8 - 8.5
	II	Average Range	119 68.0 - 202	94.[illegible] 33.3 - 223	293 61.0 - 1040	144 36.0 - 458	[illegible].3 7.67 - 49.8	13.1 2.24 - 23.1	22.4 6.46 - 48.9	12.2 2.00 - 22.7	8.56 0.75 - 14.6	0.16 tr - 0.66	0.76 0.03 - 3.59	23.6 19.0 - 26.0	7.33 6.99 - 7.53	4.8 1.5 - 12.8
	III	Average Range	250 126 - 480	191 113 - 289	804 277 - 2030	412 154 - 824	45.3 12.8 - 74.8	16.6 8.50 - 26.1	44.1 12.2 - 72.[illegible]	15.4 7.70 - 25.4	9.63 3.34 - 14.1	0.08 0.05 - 0.13	1.23 0.29 - 3.55	13.6 9.6 - 18.9	4.49 7.13 - 7.77	2.6 0.8 - 5.0
Effluent	I	Average Range	3.6 1.2 - 7.4	9.2 6.1 - 14.9	7.9 3.1 - 13.8		12.2 8.46 - 15.8	11.6 8.07 - 14.9	1.21 0.58 - 2.78	0.59 tr - 0.98	0.44 0.27 - 0.72	0.05 tr - 0.11	10.9 7.09 - 14.3	19.6 16.0 - 25.6	7.51 7.30 - 7.70	32 - 50<
	II	Average Range	1.1 0.3 - 2.7	7.5 5.6 - 11.8	7.3 2.4 - 15.3		6.91 5.68 - 8.80	6.65 5.32 - 8.76	0.91 0.41 - 2.28	0.65 0.36 - 1.03	0.42 0.06 - 0.72	0.07 0.02 - 0.12	5.93 4.30 - 7.96	24.2 17.8 - 27.8	7.44 7.05 - 7.82	22 - 50<
	III	Average Range	3.1 0.3 - 20.2	7.5 5.4 - 14.6	6.5 2.4 - 27.1		4.46 0.95 - 9.67	4.05 0.75 - 8.55	2.24 0.51 - 9.64	2.04 0.28 - 8.52	1.38 0.16 - 5.83	0.01 0 - 1.16	2.02 0 - 6.33	11.2 7.0 - 17.2	7.23 6.78 - 7.62	- 50<

Although the average removal of soluble nitrogen in the series I (continuous aeration) was 17.1%, it rose to 75.6% in the series III (intermittent aeration). Thus the experiments proved that the removal efficiency of total nitrogen of the oxidation ditch process can be improved up to about 75% of the total nitrogen by incorporating a denitrification process in the ditch. Fig. 20 shows the relationship between the removal and influent nitrogen concentration both at the Takane plant and at Okinoshima plant. At the Takane plant, the nitrogen removal was 70 to 90% in the full nitrification mode and 60 to 80% in the nitrification control mode.

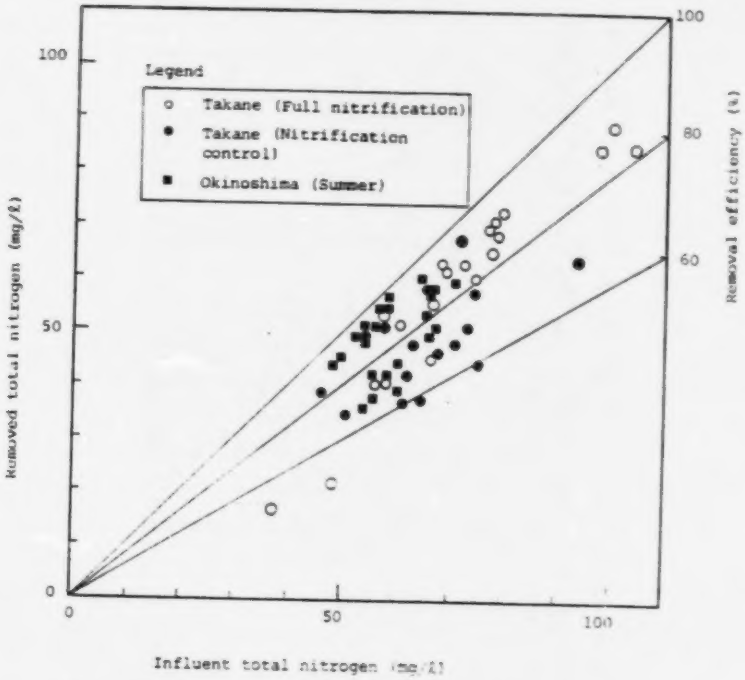


Fig. 20 Characteristic of nitrogen removal

Even in the full nitrification mode, anoxic zone existed in the ditch. Therefore, this resulted in 70 to 90% of the removal of Kjeldahl nitrogen. In the nitrification control mode nitrite and nitrate nitrogen produced by nitrification were completely denitrified, but 20 to 40% of influent Kjeldahl nitrogen remained in the effluent because of insufficient nitrification. According to the results of these two modes, an optical operating condition would be between the conditions of these two modes.

At the Okinoshima plant, the nitrogen removal 80 to 95% during summer and autumn, whereas the nitrogen removal declined to 70% to 80% in winter because of both insufficient nitrification and denitrification. Though the operating conditions did not change throughout the investigations (Fig. 8), there were signifcant difference of DO concentration among seasons, 0.2 to 0.3 mg/ℓ in summer, 0.7 to 1.2 mg/ℓ in autumn and 0.6 to 3.5 mg/ℓ in winter. In addition to the decrease of the nitrification rate in winter, the change of DO profile in the ditch, namely, decrease in anoxic zone, caused the lower removal efficiency of total nitrogen.

4. OTHER PROPERTIES

4.1 sludge Settleability

Settling of activated sludge differs depending on solid concentration. Sludge settles with clear interface being formed when the solid concentration is high, while causing hindered setting with unclear interface when the solid concentration is lower. Fig. 21 shows the relationship between MLSS and settled sludge volume after 30 minute-settling (SV_{30}) of mixed liquor or its diluted liquor. The SV_{30} is measured as sludge volume (in percentage) settled after leaving the activated sludge full of 1 litter cylinder for 30 minutes (Analytical Methods for Sewage, Japan Sewerage Works Association). Fig. 21 indicates that the group causing hindered setting has MLSS under 3000 mg/ℓ and SV_{30} within the range of 20 to 45%. On the other hand, the group forming a clear sludge interface as soon as settling starts has MLSS over 2000 mg/ℓ and SV_{30} within the range of 70 to 99%. The oxidation ditch process is normally operated under the condition that MLSS is over 3000 mg/ℓ, thereby causing settling of activated sludge with an interface being formed remarkably slow.

To improve sludge settleability, an experiment was made to investigate the effect of slow stirring with a picket fence set in the acrylic settling tower (ø200, H1200 mm). The result of the experiment is shown in Fig. 22. The figure indicates that the setting velocity of the interface formed in the activated sludge was greatly improved owing to the picket fence. This is because water paths are created due to stirring by the picket fence, thereby helping ascent of water in sludge, and improving settleability of the sludge interface.

Based on the study by the Public Works Research Institute, Fig. 23 shows the relationship between sludge consolidation ratio obtained by the picket fence method (represented by SV_{30-P}) and sludge consolidation ratio in the final settling tank (SV_S = MLSS/return sludge SS). The figure shows that SV_{30-P}/SV_S is below 1.0 in almost all cases, and indicates that the picket fence offers more consolidated sludge than that in the final settling tank.

With the above mentioned reason, it is desirable to provide a picket fence in circular shaped final settling tank for improvement of sludge settleability.

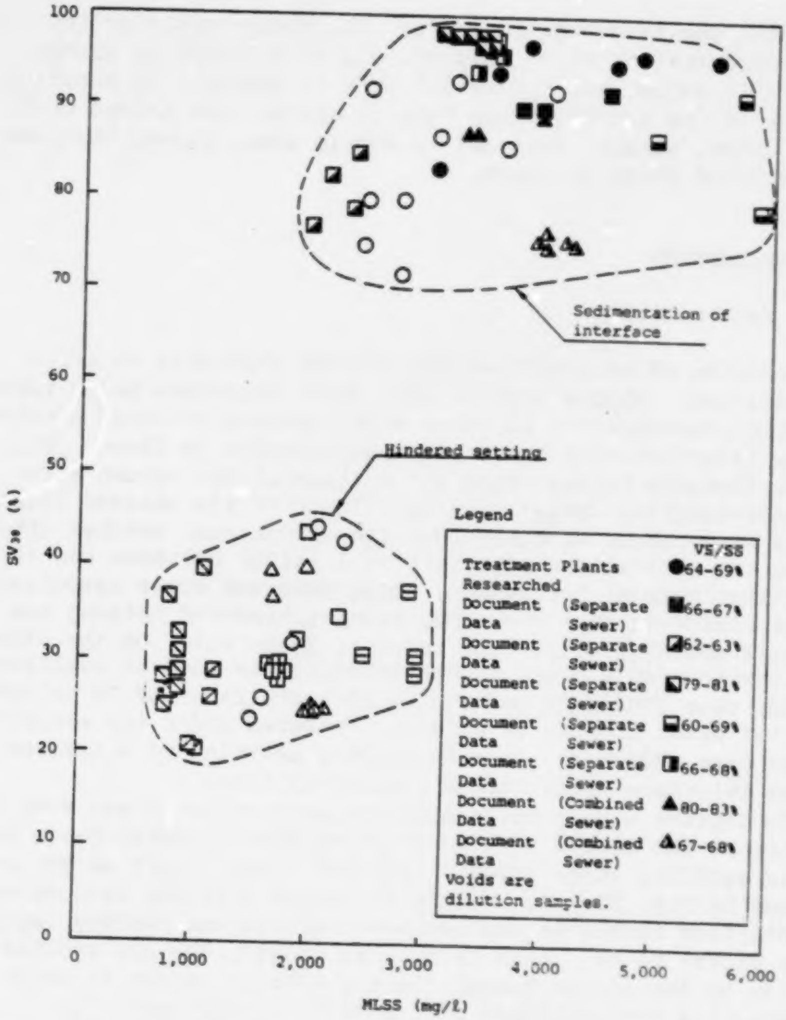


Fig. 21 Relationship between MLSS and SV_{30}

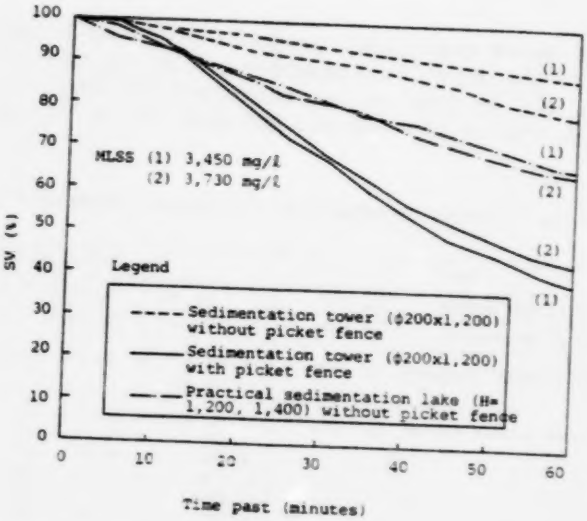


Fig. 22 Improvement of settleability by picket fence

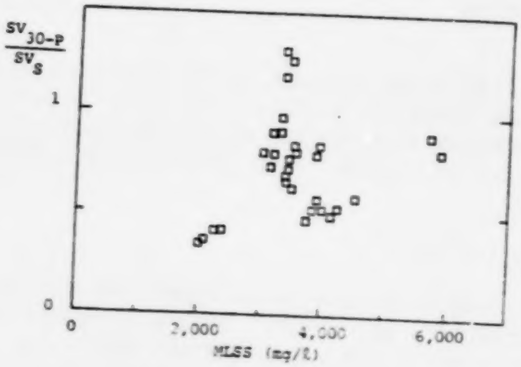


Fig. 23 SV_{30-P}/SV_S and MLSS

4.2 Sludge Production

At the pilot plant of the Public Works Research Institute, solid balance has been measured by experiment (continuous system) with a sludge retention time of 56 days both in summer and winter. The volatile suspended solids (VSS) of the influent was 70 to 75% of the suspended solids (SS). The experiment indicates that the sludge production was about 80% (in summer) or about 75% (in winter) of the removed BOD, and about 60% (in summer) or 75% (in winter) of the removed SS. Another experiment in autumn for 3 months without drawing out excess sludge shows that the sludge production was about 50% of the removed BOD and abut 70% of the removed SS.

Table 12 shows the sludge production during 3 months in autumn at Takane plant. The volatile suspended solids in the influent at Takane plant was 81% (62 to 89%) of the suspended solids. The sludge production was 73% of the removed BOD and 75% of the removed SS.

Table 12 Soudge production at Takane treatment plant

Item	Calculated value
Quantity of BOD removal ① (kg)	57.3 kg/day x 63 (days) = 3,610 kg
Quantity of SS removal ② (kg)	53.1 kg/day x 63 (days) = 3,534 kg
Quantity of MLSS increased ③ (kg)	(4,030 - 3.48) mg/ℓ x 1,320 m^3 = 726 kg
Quantity of excess sludge ④ (kg)	10,300 mg/ℓ x 36 m^3 = 371 8,980 x 54 = 485 8,170 x 40.5 = 331 10,500 x 27 = 284 11,900 x 19 = 226 6,800 x 32.4 = 220 Total 1,917 kg
Sludge production ⑤ = ③ + ④ (kg)	2,643 kg
⑤ / ① (kg/kg)	2.643 / 3.610 = 0.732
⑤ / ② (kg/kg)	2.643 / 3.534 = 0.748

The oxidation ditch process, when used to treat the wastewater which has VSS/SS about 75 to 80% (similar to ordinary domestic wastewater), enables to keep the sludge production at about 70% of the removed SS, which is smaller than that of the conventional activated sludge process.

4.3 Oxygen Consumption Rate

Oxygen consumption rate is an important factor for determination of the required quantity of oxygen. Oxygen consumption rate is influenced by BOD and nitrogen in the mixed liquor, MLSS, and water temperature.

Fig. 24 shows the relationship between specific oxygen consumption rate and water temperature in the ditch. When NH_4^+-N is over 1 mg/ℓ, the specific oxygen consumption rate depends on oxidation of BOD, nitrification, and sludge endogenous respiration. In this case, the rate constant is under strong influence of water temperature: 2 mg-O_2/g-MLSS/hour when water temperature is 10°C, and 4 mg-O_2/g-MLSS/hour when 20°C. The rate constant measured after rinsing the sludge depends on sludge endogenous respiration. In this case the influence of water temperature is small: 0.5 to 1.0 mg-O_2/g-MLSS/hour when water temperature is within the range from 10 to 30°C. Nitrification was completed when NH_4^+-N is under 1 mg/ℓ, which seems to reduce the consumption rate. In this case, too, the influence of water temperature is smaller compared with the case where NH_4^+-N is larger.

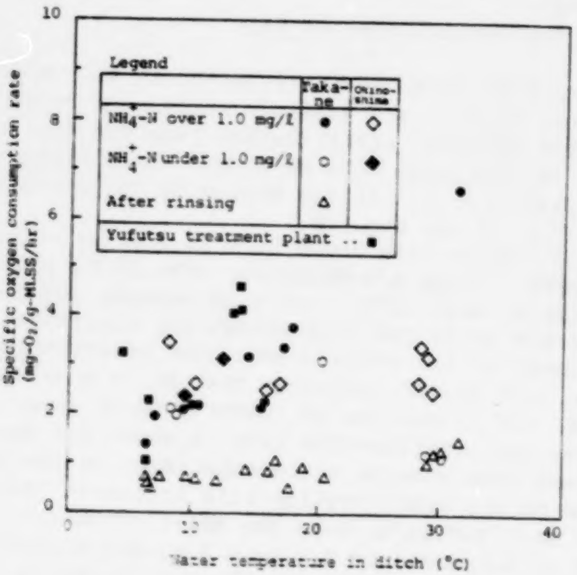


Fig. 24 Water temperature vs. oxygen utilization rate constant

5. CONSIDERATIONS ON DESIGN

When designing the oxidation ditch process, the following need be considered.

5.1 Recovery and Supply of Alkalinity

In the oxidation ditch process, which is the treatment process requiring a long sludge retention time nitrification usually takes place, which consumes 7.14 mg alkalinity to oxidize NH_4^+-N of 1 mg to NO_3^--N. Accordingly, pH of the mixed liquor falls, whereas the most appropriate condition for nitrifying bacteria is neutral to weak alkali. The fall of pH results in deterioration of nitrification rate. Therefore, countermeasure to prevent fall of pH is required for the influent with low alkalinity.

As a method of such countermeasure, recovery of alkalinity by biological denitrification shall be suggested. Reducing 1 mg of NO_3^--N into nitrogen gas through denitrification produces alkalinity of 3.75 mg. Estimates (calculated values) of alkalinity balance considering biological nitrification and denitrification, as shown in Fig. 19, conforms to the actual values, which proves the relationship mentioned above. Generally, buffer capacity becomes quite weak when alkalinity of water is below 50 mg/ℓ, thereby causing fall of pH. Thus, it is desirable, with consideration to nitrogen concentration (about 30 mg/ℓ) and alkalinity (about 200 mg/ℓ) of the Japanese general wastewater, to provide an anoxic portion so as to recover alkalinity.

5.2 Nitrification and Denitrification Rate

To achieve biological denitrification in the oxidation ditch process, aerobic portion and anoxic portion need be set spatially or timed. For this setting, nitrification rate and denitrification rate to be considered at designing must be known. Nitrifying and denitrifying rates at the practical facilities employing the oxidation ditch process are shown in Fig. 16 and Fig. 18.

The nitrification rate shown here is smaller compared to that of the NLRB process (1.6 mg-N/g-MLSS/hour when 20°C, about 0.5 mg-N/g-MLSS/hr when 10°C). This is because NH_4^+-N of the mixed liquor is low and DO is not high enough for nitrification (about 1.5 mg/ℓ or more) in the whole ditch. The relationship between nitrification rate and DO indicates that DO is a critical factor over nitrification rate in the low DO (under 1 mg/ℓ) zone. When DO is 0.5 mg/ℓ, then the nitrification rate is about 0.5 mg-N/g-MLSS/hr. The result obtained from a bench scale experiment on the effect of ammonia concentration on the nitrification rate indicated that the nitrification rate was 2.6 mg-N/g-MLSS/hr when the NH_4^+-N concentration was 10 to 15 mg/ℓ, and it decreased to 0.2 to 0.4 mg-N/g-MLSS/hr when NH_4^+-N became under 5 mg/ℓ. Accordingly, the nitrification rate of 0.2 to 0.4 mg-N/g-MLSS/hr will be adopted as a reasonable value.

The denitrification rate is also smaller compared to that of the NLRB process (about 1 mg-N/g-MLSS/hr at 20°C, and about 0.5 mg-N/g-MLSS/hr at 10°C). The reason of this seems to be the low soluble BOD. The denitrification rate in the NLRB process could be as low as 0.35 mg-N/g-MLSS/hr (at 10 to 25°C) in the nitrifying tank because of a low soluble BOD in it which is only 1/2 to 1/6 of that in the denitrifying tank. For the oxidation ditch process, denitrification rate of 0.1 to 0.2 mg-N/g-MLSS/hr will be adopted since the available BOD in the mixed liquor is low (about under 10 mg/ℓ).

5.3 Oxygen Supply

The quantity of oxygen required in the oxidation ditch process is decided with consideration to consumption of oxygen by oxidation of BOD, nitrification, and endogenous respiration. This could be expressed in the following formula:

$$\text{Required quantity of oxygen (kg-}O_2\text{/day)} = \left\{ \begin{array}{l} \text{quantity of removed BOD (kg/day)} \\ - \text{ denitrified nitrogen (kg/day)} \times m \\ + \text{ nitrified nitrogen (kg/day)} \times n \end{array} \right\} \times a + b \times \Sigma \text{MLSS}$$

Where;

a: Quantity of oxygen required for unit BOD removal, which is about 0.5 - 0.7.

m: Quantity of BOD required for unit denitrified nitrogen, which is about 2.

b: Quantity of oxygen consumed by endogenous respiration. 24 mg-O_2/g-MLSS/day will be appropriate when the oxygen consumption rate after rinsing shown in Fig. 24 is used.

n: Quantity of oxygen required for nitrification. It is 4.57 when nitrification is to be expressed by the equation: $NH_4^+ + 2O_2 \longrightarrow NO_3^- + 2H^+ + H_2O$.

When calculation is done using the following conditions (removed BOD: 180 mg/ℓ, denitrified nitrogen: 20 mg/ℓ, MLSS: 3000 - 5000 mg/ℓ, nitrified nitrogen: 24 mg/ℓ, and hydraulic retention time in the ditch: 24 - 48 hours), the oxygen consumption rate will be 2.4 - 3.3 mg-O_2/g-MLSS/hr. These values, compared with the measured values in Fig. 24, seem to be appropriate.

Based upon the above mentioned theory, the required quantity of oxygen will be decided considering denitrification and safety factor of the design.

5.4 Control and Adjustment of Aerator

Nitrification and denitrification have much to do with DO profile in the ditch. Required quantity of oxygen in the ditch fluctuates according to the changes in influent, water temperature, and MLSS. For example, oxygen consumption rate goes up as water temperature rises. Accordingly, quantity of oxygen supplied by the aerator need occasionally be increased to secure aerobic area required. To prepare for such case, the number of rotations of the aerator and the number of running units need be designed controllable and the dip depth adjustable.

6. CONCLUSION

Through researches with pilot plant and full-scale facilities in service, it has been proved that removal of BOD and suspended solids could be done stably irrespective of seasonal, daily, and hourly fluctuations. It has also been turned out that anoxic portion set for the oxidation ditch process effectively helps to remove nitrogen, to recover alkalinity, and to decrease required quantity of oxygen (i.e., decrease of demanded energy). For sufficient nitrogen removal, it is necessary to provide the hydraulic retention time of 40 - 48 hours for the ditch capacity.

PLANNING AND DESIGN MANUAL
FOR
SMALL-SCALE SEWERAGE SYSTEM

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

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1. BACKGROUND AND STATUS QUO OF SMALL-SCALE SEWERAGE SYSTEM

1.1 Background

Population served by sewerage system in Japan was 33% in 1983. One hundred (100) years had past from 1883 when it began to lay sewers in the ground. Before the World War II and for ten years after the War, however, preparation of industrial basis such as roads and railways was mainly performed while only big cities dare to achieve sewerage works. During 1950's and 60's economy grew quite rapidly, and at the same time it caused environmental pollution. The needs for environmental pollution control had promoted increase in the spending for sewerage works with the help of financial advantage thanks to the high economic growth. In recent years, in spite of severe financial condition, 1.6 to 1.8 trillion yen per year (0.6 - 0.8% of GNP) is being invested for the sewerage works.

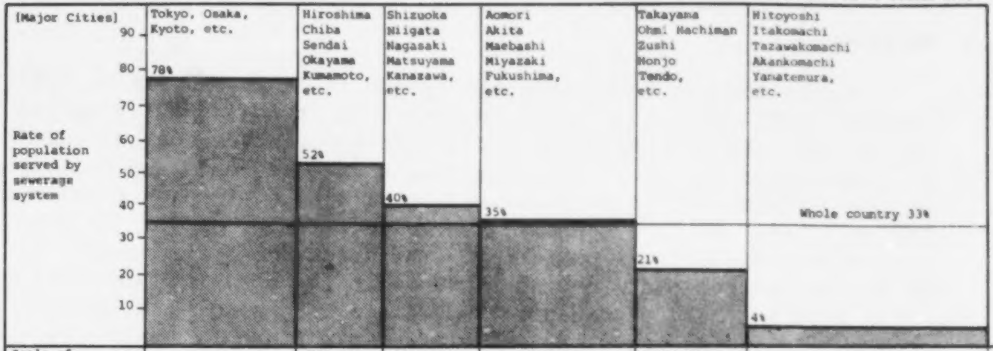
Deviation of population served from the national average is quite large depending on areas or cities with their scale. Fig. 1 presents the ratios of population served by sewerage system of Japanese cities classified according to population ranks.

A city having population over one million, such as Tokyo and Osaka, has been served very well, about 80%. This owes to the fact that the national and the local governments jointly made effort to promote sewerage works since population and industries had centered therein causing severe environmental pollution. While population served in large cities is relatively high, that in small cities with population of less than 50 thousand is only 4% on the average. The total population in the municipalities with population less than 50 thousand is about 36.6 million, which accounts for about one third of the Japanese population. Most of these people are not receiving benefit of the sewerage system.

As for the number of cities carrying out sewerage works, it is all of the cities having population over three handred thousand, and 80 to 90% of the cities having population over fifty thousand. However, talking about cities having population under fifty thousand, it is only 512 cities (18%) out of 2,834 that have started sewerage works. Further, it is only 157 cities that have sewage treatment plants in operation.

This does not mean that the cities with population under fifty thousand have their environments well conserved and have no necessity for sewerage preparation. According to the "Survey on the Opinions of Mayors Concerning Sewerage" done by the Japan Sewage Works Association in 1984, the mayors of smaller municipalities answered that the quality of public waters such as rivers, lakes, and coastal seas has become worse than that ten years ago. Especially the mayors of the cities each having population of ten to thirty thousand, who answered that the quality of their rivers, lakes, and coastal seas deteriorated show the highest rate. Fig. 2 and Fig. 3 show their answers concerning the quality of public waters in their cities.

<as of the end of fiscal 1983>



Scale of population	Over 1,000,000	500,000 - 1,000,000	300,000 - 500,000	100,000 - 300,000	50,000 - 100,000	Under 50,000	Total
Total population (unit: 10,000)	2,321	626	1,443	2,360	1,517	3,665	11,932
Population served by sewerage system (unit: 10,000)	1,820	323	581	816	311	136	3,987
Total number of cities	10	10	38	141	223	2,834	3,256
Number of cities achieving sewerage works	10	10	38	132	183	512	885
Number of cities having sewerage in service	10	10	37	117	122	159	455

Note: (Survey by the Ministry of Construction)

1. Details of the total number of cities are as follows: 652 cities, 1,993 towns, and 611 villages.

Fig. 1 Sewerage in service in Japanese cities classified by population rank

Table 1 shows the priorities (at present and after 10 years) among preparations within the limited budget, of public properties such as repetments of rivers, roads, and sewerage, etc. The table tells that every municipality having population over thirty thousand and of higher ranks are putting priority on sewage works both at present and after 10 years. On the other hand, the priority given to sewage works in cities having population under thirty thousand is the fourth (4th) at present. However, the first (1st) priority is given to the sewage works after ten (10) years.

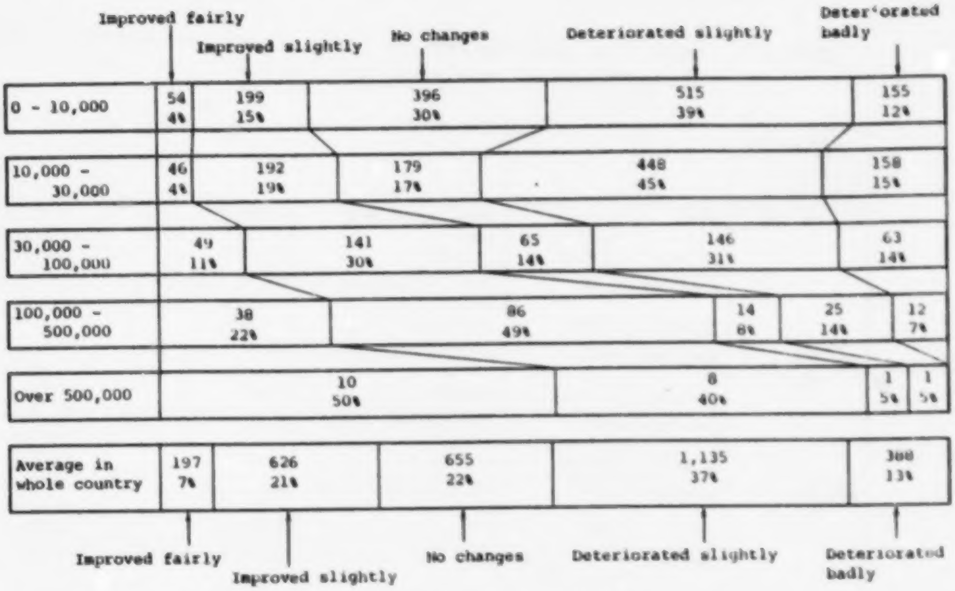


Fig. 2 Water quality of rivers and lakes compared to that 10 years ago

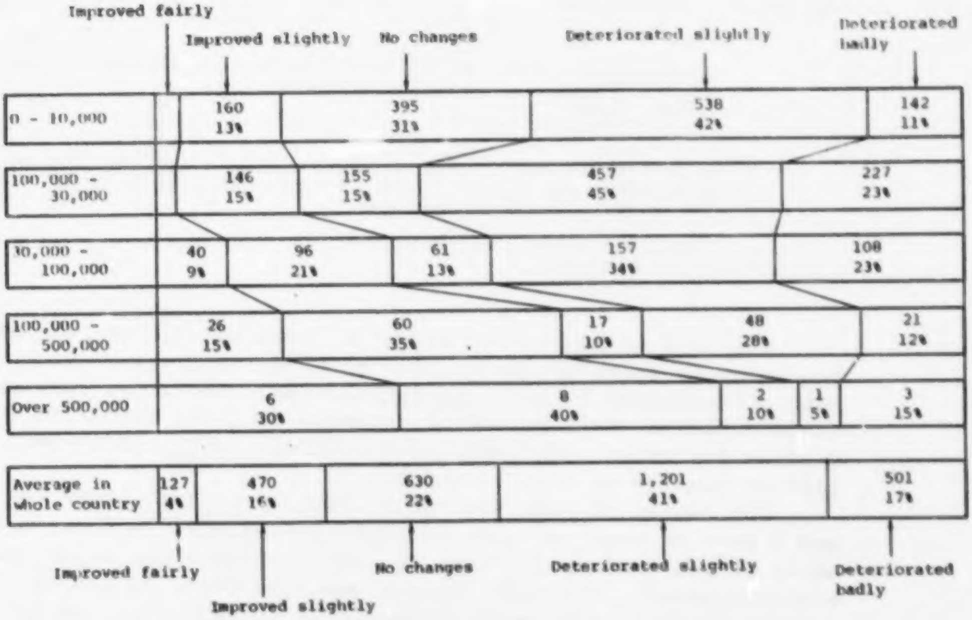


Fig. 3 Water quality of small streams compared to that 10 years ago

Table 1 Best 5 of priorities in infrastructure construction (classified by scale of population)

Priority / Scale of population	1	2	3	4	5
0 - 30,000	Road (8,833)	Cultural facilities (4,839)	Park, green area etc. (3,427)	Sewerage (3,096)	Welfare and medical treatment facilities (2,643)
	Sewerage (6,953)	Road (5,070)	Park, green area etc. (4,214)	Welfare and medical treatment facilities (3,429)	Cultural facilities (3,042)
30, 000 - 100,000	Sewerage (1,571)	Road (1,335)	Cultural facilities (1,027)	Park, green area etc. (826)	Welfare and medical treatment facilities (515)
	Sewerage (1,864)	Road (1,139)	Park, green area etc. (1,011)	Cultural facilities (694)	Welfare and medical treatment facilities (652)
100,000 - 300,000	Sewerage (530)	Road (372)	Cultural facilities (275)	Park, green area etc. (257)	Welfare and medical treatment facilities (178)
	Sewerage (497)	Park, green area, etc. (336)	Road (317)	Cultural facilities (216)	Welfare and medical treatment facilities (197)
300,000 - 500,000	Sewerage (165)	Road (87)	Cultural facilities (60)	Park, green area etc. (55)	Revetment of watercourses (45)
	Sewerage (145)	Park, green area, etc. (85)	Road (64)	Welfare and medical treatment facilities (62)	Cultural facilities (59)
Over 500,000	Sewerage (84)	Road (38)	Park, green area etc. (35)	Welfare and medical treatment facilities (29)	Cultural facilities (28)
	Sewerage (84)	Park, green area, etc. (43)	Welfare and medical treatment facilities (30)	Traffic facilities (29)	Road (26)

Note: Values in parenthesis ( ) represent total evaluation point. The total evaluation point was calculated with 5 points for the first priority, 4 for the second, 3 for the third, 2 for the fourth, and 1 for the fifth.

Reference from the "Survey on the Opinions of Mayors Concerning Sewerage" as of 1984 by Japan Sewerage Works Association.

The rate of population served by sewerage system remains at a very low level in spite of the quality of rivers and water channels having been deteriorated in small municipalities. Therefore such small municipalities shall positively promote sewerage construction to meet desires for urban life style such as flushing toilet as well as to protect quality of public waters.

The center of sewerage works which were mostly done for big cities and a part of other cities, has been moving over to local mid and small scale cities through the time the whole country has experienced the high economic growth. In near future, sewerage works in smaller municipalities and/or small communities around urbanized areas shall become to hold a great ratio.

The Japanese sewerage technology has been developed as sewage works achieved in big cities and stored up as well. The most appropriate technology and system have been pursued for big cities. It is comparatively easy to secure personnel with high technology thereby introducing facilities which are benefitted with high technology rather economically thanks to the scale merit. However, application of such technology to small-scale sewerage system may cause many problems. For example, it will be difficult to secure high technical engineers even though treatment facilities employing high technology is established. In addition, cost per capita will be high owing to the scale demerit. As for pipe laying works, the cost will also be high because of lower population density which results in longer pipe per capita. These factors cause initial cost per capita for sewage works to be increased. Further, such increase makes it more difficult for minicipalities to start sewage works since most of them are financially weak.

To meet the needs of sewage works in municipalities, establishment of effective planning and designing process for small scale sewage system has been requested. Therefore, the Ministry of Construction drew up the "Planning and Design Manual for Small-Scale Sewerage System (draft)". This manual (draft) is now tentatively applied so as to make its contents more accomplished. Along with preparation of this manual, the Japan Sewerage Works Agency entrusted by the Ministry of Construction is performing researches and development of facilities which are appropriate for small-scale sewerage system.

The outline of "Planning and Design Manual for Small-Scale Sewerage System (draft)" and the researches and development achieved by the Japan Sewerage Works Agency are described separately in Chpater 2 and Chapter 3.

1.2 Status Quo

The sewerage systems for small communities include public sewerage systems (under jurisdiction of the Ministry of Construction), agricultural community wastewater treatment works (under jurisdiction of the Ministry of Agriculture and Fishery), and local community night soil treatment works (under jurisdiction of the Ministry of Health and Welfare). (See Table 2). In addition, joint treatment facilities managed by the residents of housing estates without any subsidy could be listed.

Public sewerage systems can be classified into two: public sewerage systems in a narrow sense and specific environment protection public sewerage works. The narrow-sense public sewerage systems, whose scale is not specifically specified, are applied to urbanized areas. Specific envornment protection public sewerage systems are to be applied to outside of city palnning areas or to urbanization control areas having population over 1,000 but no more than 10,000.

Table 2 Classification of small scale sewage treatment works

Classification	Population planned for treatment	Object area	Regulation Law	Jurisdictional Ministry
Public sewerage (in narrow sense)	-	Urbanized areas	Sewerage law	Ministry of Construction
Specific environment protection public sewerage works	1,000 - 10,000	Other than city planning areas or urbanization control areas	Sewerage law	Ministry of Construction
Agriculatural, community waste-water treatment works	1,000 or less	Agriculture promotion areas	Waste Disposal and Public Cleasing Law (Construction Stnadardization Law)	Ministry of Agriculture and Fishery
Local community hight soil treat-ment works	101 - 30,000		Law for Disposal of Waste and Cleaning	Ministry of Health and Welfare

Accordingly, small scale sewage works are mostly performed as specific environment protection public sewerage. The specific environment protection public sewerage is divided into two: nature protection sewerage and rural village sewerage. The nature protection sewerage is prepared to protect the quality of water of rivers, lakes, and reservoirs in national and seminational parks. The other rural village sewerage is prepared to avoid pollution of the public waters for farming due to domestic wastewater from such villages. The agricultural community wastewater treatment works are applied to the area having population scale under 1,000. At the end of 1983, nature protection sewerage works have been started at 34 places and rural village sewerage works at 52 places in the whole country. As for public sewerage works in a narrow sense, 53 municipalities having population less than ten (10) thousand have get started by the end of 1983.

Table 3 shows the treatment plants in operation by the end of 1982 classified by capacity and by treatment processes. As for the treatment process employed at the secondary treatment plants, most of those handling 5,000 m^3/day or more adopts conventional activated sludge process or step aeration process. The treatment process at each plant handling less than 5,000 m^3/day is principally the conventional activated sludge process, which is employed by one second (1/2) of the secondary treatment plants. However, variety has been appearing in methods such as extended aeration process, oxidation ditch, and rotating biological contactor process. This is because the small scale treatment plants require processes which are easier to oprate and maintain. Treatment processes tend to vary more in the treatment plants under construction or planned, which means that each numicipality is trying to employ the technology that is appropriate for its own local conditions.

Table 3 Treatment plants classified by capacity and treatment process

(As of the end of fiscal 1982)

Treatment process / Design daily maximum dry weather flow (unit: 1,000 m^3/day)	Primary treatment	Semi-secondary treatment		Secondary treatment							Total
	Plain sedimentation process	High-rate trickling filter process	High-rate aeration process	Conventional activated sludge process	Stepped aeration process	Extended aeration process	Contact stabilization process	Pure Oxygen Aeration process	Oxidation Ditch	Rotating Biological contactor process	
Under 5	2	2	2	37	5	12	1	2	6	7	76
5 - 10	1	3		45	6	1				3	61
10 - 50	2	6	14	121	13			1		2	150
50 - 100		1	3	69	23					1	96
100 - 500		1		88	21			2			112
Over 500				10	9						19
Total	20	13	19	370	77	13	1	5	6	13	522

Notes: 1. When two (2) types of treatment processes exist at a treatment plant, the method treating greater water flow is reflected in this table.

2. Details of the total 522 treatment plants:

Public sewerage	460 plants
River Basin-wide sewerage	41 plants
Specific public sewerage	8 plants
Specific environment protection public sewerage	13 plants

Agricultural village wastewater treatment works which are under jurisdiction of the Ministry of Agriculture and Fishery are to be achieved for treatment of wastewater to prevent irrigation channels and other waterways from being polluted. Such works are applied in agriculture promotion areas, fishing port areas, and forestry promotion areas in principle. Each treatment plant has the scale below 1,000 people in principle. One hundred and thirty-one (131) districts have begun works by 1983 and thirty-nine (39) of them had already started servicing. Approximately 70% of the treatment process employed for the plants which are already in service is soil covered type contactor oxidation process. The remaining 30% is split approximately equally among methods such as the rotating biological contactor process, extended aeration process, and conventional activated sludge process.

The local community night soil treatment works are under jurisdiction of the Ministry of Health and Welfare. The works are constructed based on the plan concerning disposal of waste provided by munipality. Domestic wastewater including excrement from a housing development and so on is treated by such facility that should be managed by local public bodies. The government subsidy is to be applied for the works which meet a certain requisite (treatment capacity should be over 101 people but no more than 30,000). According to the white paper on welfare, the local community night soil treatment works have been established at 869 places in the whole country by the end of fiscal 1979. Most of them have employed the extended aeration process or conventional activated sludge process. The criteria of the final effluent from these facilities is, based on the structural guideline, BOD under 30 mg/ℓ, SS under 70 mg/ℓ, and coliform count 3,000 organisms/cm^3. This criteria is rather loose compared to the criteria of Sewerage Law applied for the secondary treatment (BOD under 20 mg/ℓ, SS under 70 mg/ℓ, and coliform count 3,000 organisms/cm^3).

2. PLANNING AND DESIGN MANUAL FOR SMALL-SCALE SEWERAGE SYSTEM (DRAFT)

2.1 Details of Manual Preparation

As the design manual for sewerage system in Japan, the "Design Criteria for Sewerage System" published by the Japan Sewage Works Association has widely been used. Since the sewerage systems in Japan have been implemented mainly in large cities, various preconditions such as the population density, road conditions, underground facility status, load of the sewage treatment plant and fluctuation of quantity and quality over time are assumed mainly on the basis of the conditions in large cities. As described in the preceding chapter, however, implementation of sewerage systems in small-scale cities in provincial districts is becoming an important theme for the future. Therefore, the necessity of another manual for small-scale sewerage system in small cities is being recognized. In small cities socioeconomical conditions such as the city characteristic, surrounding environment and financial standing are generally different from those in large cities, aside from the above technical conditions. It is natural that the sewerage system planning and implementation methods are somewhat different. For smooth, efficient execution of small-scale sewerage system construction works which will increase much more in the future, the Ministry of Construction has determined to prepare a new planning and design manual and entrusted the Japan Sewerage Works Association with the preparation job in 1981. Japan Sewage Works Association formed a manual preparation committee consisting of the experts of the Ministry of Construction, the Japan Sewage Works Agency, prefectural governments and municipal governments, and finalized the "Planning and Design Manual for Small-Scale Sewerage System (Draft)" in October 1984.

The main features of this draft manual are explained below in comparison with the existing design criteria. The Design Criteria for Sewerage System is hereafter called the "Design Criteria", and this Planning and Design Manual for Small-Scale Sewerage System the "Small Facility Design Manual".

2.2 Sewerage System Plan

(1) Scope of application

As a rule sewerage system implementation has so far been executed only in DIDs (densely inhabited districts) defined as the communities each with a pepulation over 5,000 persons and a population density over 40 persons/ha. Therefore, the Design Criteria assumes sewerage systems in such communities. On the other hand, the Small Facility Design Manual assumes the use for planning and design of sewerage systems in districts smaller in scale, or where the population is less than 5,000.

(2) Target year for planning

The Design Criteria assumes about 20 years later. Sewerage system will have to be usable under possible changes in the future because sewers, once laid, are very hard to be reconstructed.

In the Small Facility Design Manual, the current conditions are generally adopted because large population change in the object district is not generally possible.

(3) Storm water drainage

Sewerage systems in Japan are generally planned as separate sewer systems. The Small Facility Design Manual assumes use of existing drainage facilities as a rule for storm water drainage and does not include storm sewer planning except for a special case.

(4) Facility structure

The Design Criteria assumes semipermanent reinforced concrete construction for facilities. The Small Facility Design Manual assumes, simple facility construction as far as possible. It lays stress on the economy of construction investment and adaptability to technical innovation and does not insist on semipermanent construction. For these purposes, it recommends use of prefabricated structures and machinery and equipment on the market while trying standardization of facilities.

(5) Basic thinking about operation and maintenance

The Small Facility Design Manual basically assumes no full-time operator for the control of the pumping station and treatment plant, but the patrol once or twice a week and adoption of the remote alarm system. Therefore, the pumping station as well as the facilities in the sewerage treatment plant are to be operated automatically.

2.3 Sewer Design

(1) Fluctuation of sanitary sewage flow

The Design Criteria assumes the ratio of the design hourly maximum flow to the design daily maximum flow to be 1.3 to 1.8.

In a small-scale sewerage system, however, the hourly flow fluctuation is even greater. Fig. 4 shows the water supply data. When the daily water supply is in the scale of 1,000 m^3/day, the ratio of the hourly maximum to the daily maximum is 2 to 3. Since some storage retention in the sewer can be expected in a sewerage system, the Small Facility Design Manual assumes the ratio of the design hourly maximum sanitary sewage flow to the design daily maximum flow to be 2.

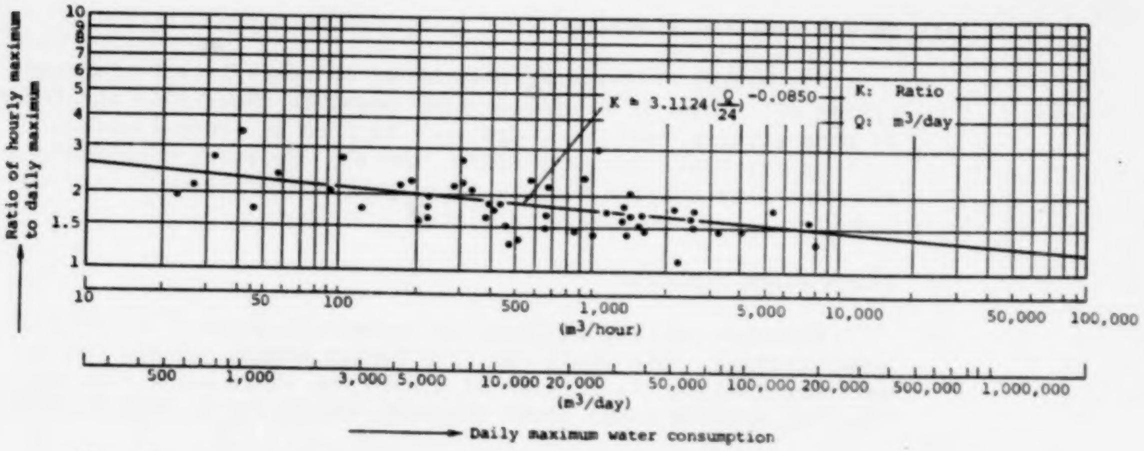


Fig. 4 Ratio of hourly maximum water consumption to daily maximum water consumption at 60 water supply districts (general residential areas) in 16 cities

(2) Sewer cross-sectional shape and minimum pipe diameter

The Design Criteria defines the sewer cross section to be mainly circular or rectangular, and the minimum pipe diameter to be 200 mm.

On the other hand, the Small Facility Design Manual recommends circular or egg-shaped pipe as a rule, and the minimum pipe diameter of 100 mm. Since the house density is small in a small-scale sewerage system, the sanitary sewerage flow in the end branch is generally small. Increasing the dragging power by decreasing the pipe diameter has been considered to be favorable. In adopting the 100 mm pipe, a verification test was conducted by the Research and Technology Development Division of Japan Sewage Works Agency as described later.

Fig. 5 shows the cross-sectional view of the egg-shaped high rigidity PVC pipe as an exmaple of egg-shaped pipes.

(3) Laying position

The Design Criteria specifies that sewer pipes should be laid under public roads and the minimum earth convering, shoudl be 1 m.

The Small Facility Design Manual does not limit the laying position to under public roads but allows laying in private land if it is favorable. It also recommends minimization of the earth covering considering the load and allows exposed sewers if necessary so as to economize the laying cost as far as possible. Fig. 6 shows the example of exposed sewers.

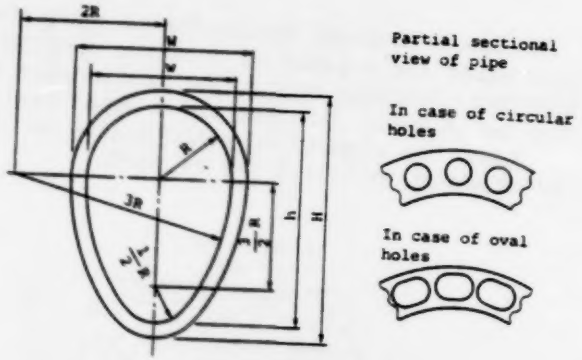

2R
W
w
R
3R
1/2 R
R/2
h
H
Partial sectional view of pipe
In case of circular holes
In case of oval holes

Fig. 5 Sectional view of high-rigidity egg-shaped PVC pipe

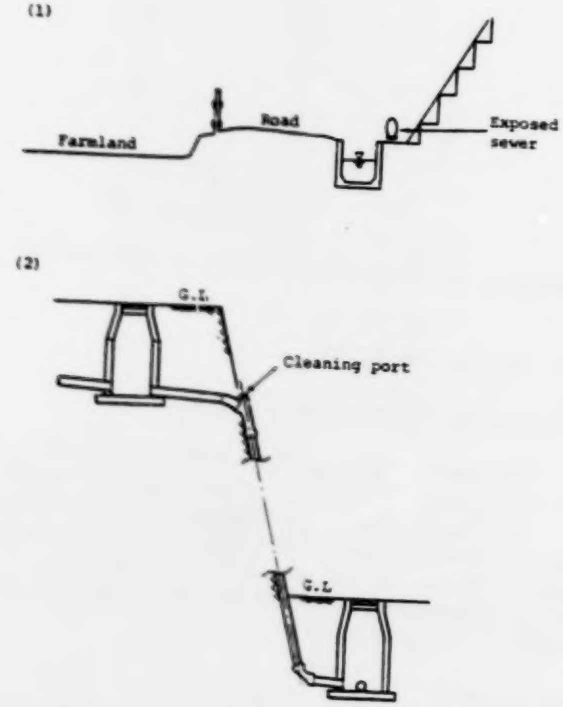

(1)
Farmland
Road
Exposed sewer
(2)
G.L.
Cleaning port
G.L

Fig. 6 Example of exposed sewer

(4) Manhole

The Design Criteria requires a manhole to be always provided at a curved portion of the sewer. The minimum inside diameter of the manhole is 900 mm.

The Small Facility Design Manual specifies that manhole may be omitted using angle pipes at a curved portion of sewer so long as it does not adversely affect the maintenance and operation. In a section using small diameter pipes whose inside diameter is less than 150 mm, the simplified manhole with an inside diameter of 500 mm as shown in Fig. 7 may be used.

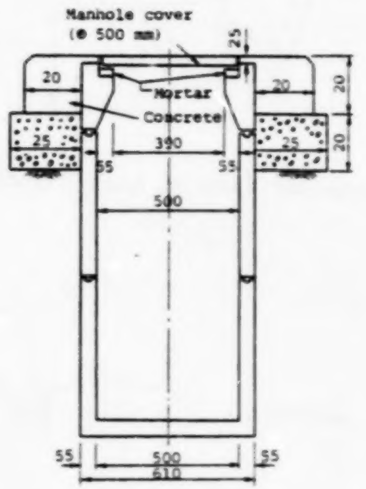


Fig. 7 Example of simplified manhole with inside diameter of 500 mm

2.4 Design of Pumping Facility

(1) Installation of pumping facility

The Design Criteria specifies gravity flow as the basic rule, and installation of the relay pump as required.

The Small Facility Design Manual evaluates the role of the pumping station more positively, and recommends pumping station establishment if it is favorable in view of the total construction and maintenance cost. It also recommends positive study of pressure sewer adoption.

(2) Structure

The Small Facility Design Manual encourages adoption of simple pumping station. Fig. 8 shows an example of simplified manhole type pumping station. The screen and grit chamber for relay pumping station are not to be provided as a rule.

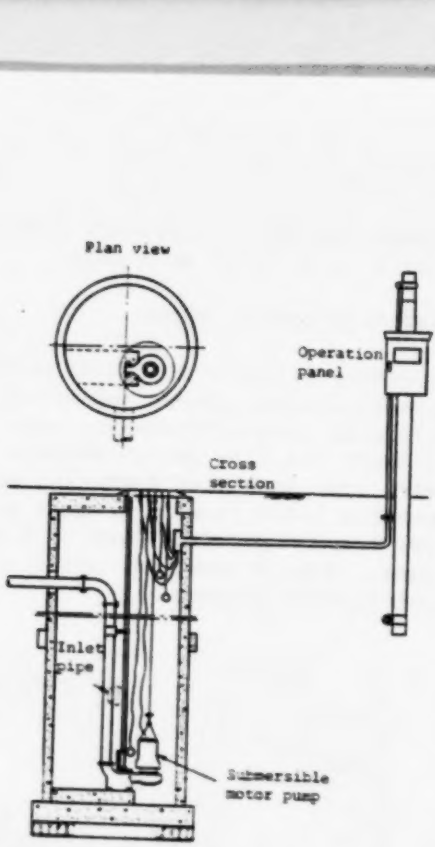


Fig. 8 Example of manhole type pumping station

(3) Pump type and minimum diameter

The Small Facility Design Manual specifies the use of detachable submersible pumps as a rule. The minimum pump diameter is 80 mm as a rule similarly as in the Design Criteria, but the use of 65 mm or smaller pumps is allowed if pumps clogging by large floating substances can be prevented.

(4) Reserve pumps

The Design Criteria specifies installation of one reserve pump in each pumping station. The Small Facility Design Manual does not specify one reserve pump installation for every pumping station but at least one reserve pump in one treatment area.

2.5 Treatment Facilities

(1) Grit chamber, screen and primary sedimentation tank

The Small Facility Design Manual defines installation of no grit chamber as a rule for simplification of maintenance and operation. Screens are provided for pump protection, but they are limited to one type as a rule. If the secondary treatment process is of the fixed-film biological reactor type, the primary settling tank or substituting fine-mesh screen is to be provided. If the secondary treatment process is of the suspended growth biological reactor type, neither the primary sedimentation tank or fine-mesh screen shall be provided as a rule.

(2) Wastewater treatment process

The Design Criteria specifies only time-tested processes like the activated sludge process and trickling filters for which the design criteria have sufficiently been established.

The Small Facility Design Manual, however, lays stress on low BOD loading type processes featuring easy control. It describes the sequencing batch reactor process and submerged fixed-film biological reactor process which have not much positive performance in the past. Fig. 9 shows an exaple of the submerged fixed-film biological reactor process.

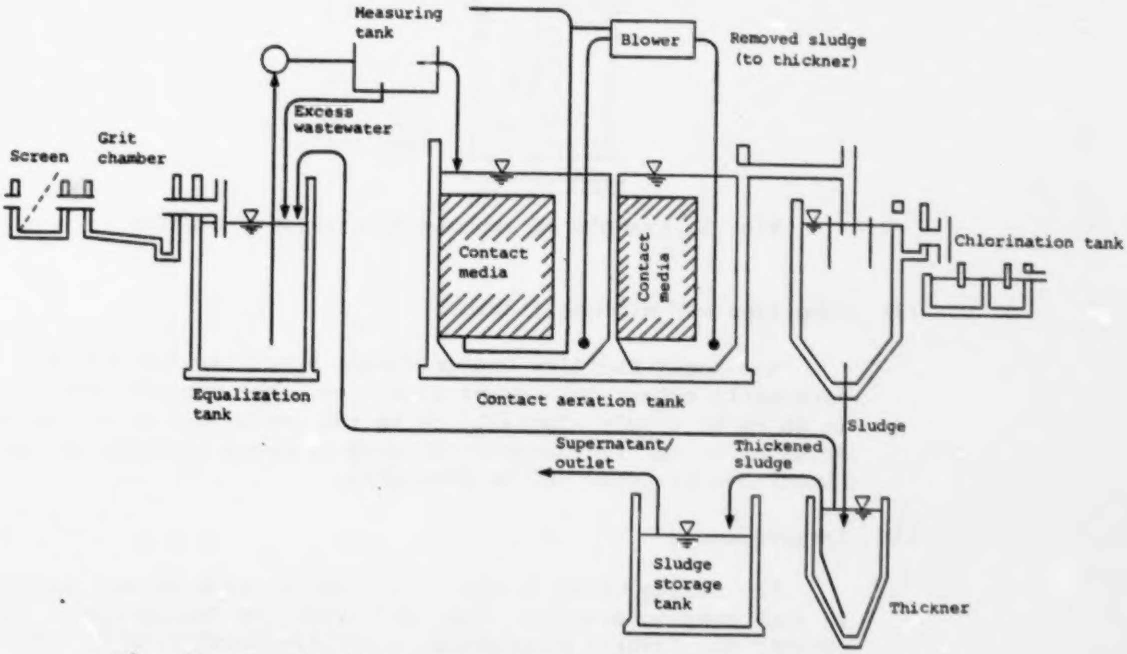

Measuring tank
Blower
Removed sludge (to thickner)
Excess wastewater
Screen
Grit chamber
Contact media
Contact media
Chlorination tank
Equalization tank
Contact aeration tank
Sludge
Supernatant/ outlet
Thickened sludge
Sludge storage tank
Thickner

Fig. 9 Example of submerged fixed-film biological reactor

(3) Sludge treatment and disposal

The Small Facility Design Manual recommends the non-heated anaerobic degestion, aerobic digestion, and sludge drying beed treatment process featuring relatively easy operation and maintenance. It also describes the centralized processing of the sludge collected from several treatment plants or the patrol maintenance system should first be studied. Utilization for agricultural purposes is especially recommended as regards sludge disposal.

(4) Instrumentation system

As described before, the Small Facility Design Manual basically assumes automatic operation for the pumping and wastewater treatment facilities. On the other hand, it recommends that the instrumentation system should be as simple as possible, mainly consisting of alarms.

2.6 Model Works of Small-Scale Sewerage System

The Small Facility Design Manual adopts new concepts having not much past performance. Model systems applied with this manual have been planned and executed at 6 places shown in Table 4. Full application of this manual will start after checking the results.

Table 4 Model municipalities applied with Planning and Design Manual for Small-Scale Sewerage System

Prefecture	Municipality	Year of adoption	Design population (whole plan)
Ibaragi	Dejima-mura	1984	1,800
Niigata	Nagaoka-shi	1984	1,000
Yamanashi	Takane-cho	1983	2,000
Gunma	Yoshioka-mura	1983	1,820
Yamagata	Haguro-machi	1978	4,000
Niigata	Sumon-mura	1978	4,100

3. RESEARCH AND DEVELOPMENT OF FACILITIES FOR SMALL-SCALE SEWERAGE SYSTEM

3.1 Outline of Development

The Japan Sewerage Works Agency has been doing researches and developments of ficilities which are appropriate for small-scale sewerage system, while the preparation of the Small Facility Design Manual by the Japan Sewerage Works Association.

The small scale sewerage must fit the natural and social conditions of local areas different from city areas. Characteristics (such as living environments and state of public facilities) of agricultural villages, representing the area to which small-scale sewerage system is to be applied, are researched before development of facilities. With consideration to the characteristics known from the research, the purpose of development has been set as follows: 1) efficient construction, 2) simple operation and maintenance, and 3) harmony with the local society. The direction of development of each facilities was studied based on such purpose.

Mian theme of this research was to improve the existing facilities and equipments and to develop new technologies. Contents of the research are described hereinafter.

3.2 Development of Collection Facility

Density of houses is low in the area to which small-scale sewerage is applied thereby requiring longer sewer per house. The grade of the sewer shall be great since sanitary sewage flow is small thereby increasing earth-covering. In addition, the number of manholes per sewer length increases because of narrow roads with many corners. Thus the collection facility for the small-scale sewerage system tends to cost more per capita. It is unavoidable to have longer sewer per house. However, it is necessary to study measures to lessen earth-covering, to decrease the number of manholes, and to hold down the construction cost.

3.2.1 Branch pipe

In a small community, very small quantity of sanitary sewage is expected within the sewer from the starting point to the point to which sanitary sewage from dozens of houses is collected. Such sewer mentioned above was defined as "branch pipe". The measure to decrease earth-covering of branch pipe is discussed below.

When designing sewer, size and grade of pipe have been decided by calculation based on steady flow of the maximum hourly flow. The Design Criteria specifies the minimum sewer size as 200 mm and the minimum velocity of flow as 0.6 m/sec. The branch pipe, in which a small flow is expected, requires a steep grade to secure such velocity of flow. Accordingly, the branch pipe need be laid deep in the ground thereby increasing the construction cost.

The sanilary sewage does not inlet continuously into the branch pipe from house drain equipments. The flow in the branch pipe need be considered as unsteady-state flow. Considering unsteady-state flow in the branch pipe the energy at the peak flow shall be used as dragging power thereby making the grade of pipe smaller. Based on this idea, we studied about the minimum size of pipe and minimum velocity. Egg-shaped PVC pipe of 100 mm and 150 mm have been studied considering connectibility with access pipe and maintainability.

(1) Hydraulic characteristics

To clarify characteristics of flow in the branch pipe, measurement using an experimental equipment and simulation via computer were performed. Fig. 10 shows the outline of the experimental equipment. On the conditions of 15 ℓ/time of toilet drain, 30 ℓ/time of laundry drain 90 ℓ/time of bathtub drain, flow in the pipe were measured at 25 m, 50 m, 75 m, and 100 m each from the draining point.

The theoretical flow was computed via electronic computer with the same condition used in the experiment. The following basic equations which are used for open channel unsteady-state flow were used as basic formula for computation:

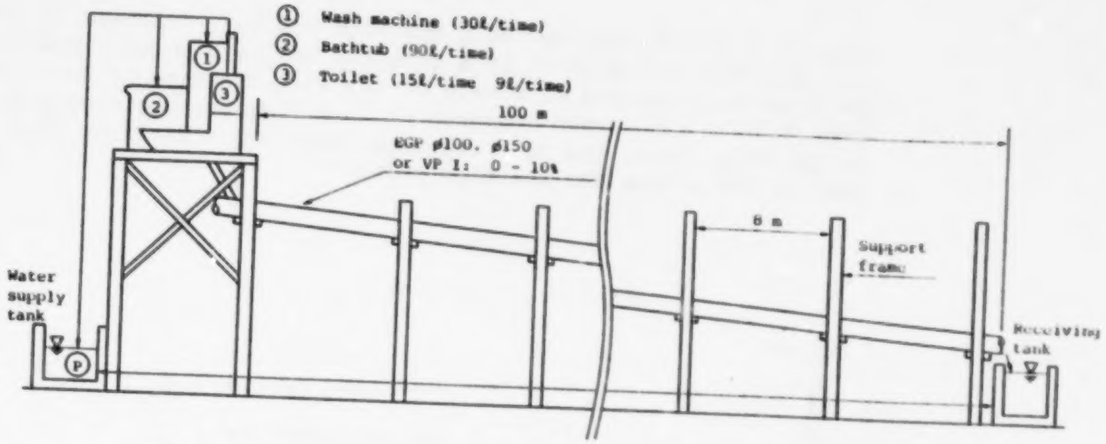

① Wash machine (30ℓ/time)
② Bathtub (90ℓ/time)
③ Toilet (15ℓ/time 9ℓ/time)
100 m
EGP ø100. ø150
or VP I: 0 - 10%
6 m
Support frame
Water supply tank
P
Receiving tank

Fig. 10 Experimental facility for studying the hydraulic characteristics of egg-shaped pipe

Equation of Continuity

$$\frac{\partial A}{\partial t} = - \frac{\partial Q}{\partial X} \quad \cdots\cdots \ (1)$$

Equation of Motion

$$\frac{\partial Q}{\partial t} = - \frac{\partial Q}{\partial X} - \frac{\partial P}{\partial X} + giA - gn^2\frac{VQ}{R^{4/3}}$$

where;
t: time
X: distance
i: slope of pipe
g: gravitotional acceleration
P: water pressure
A: flow cross section area
Q: flow rate
n: roughness coefficient of Manning
V: flow velocity
R: hydraulic radius

An example of the results of experiment and computation is shown in Fig. 11. The actual values and computed values almost agree each other, which proves that hydraulic characteristics of the branch pipe conforms to those of open channel unsteady-state flow. From the experiment and computation, the following are known about hydraulic characteristics of the branch pipe.

Decrement of the momentary maximum flow rate (ℓ/sec) by the flowing distance is greatly influenced by the quantity of drain. The more flow rate is, the longer the distnace is before the momentary maximum flow rate is decreased. For example, the flow rate of 15 ℓ drainage and 90 ℓ drainage and 90 ℓ drainage is nearly the same when measured at 15 m from the draining point. However, the momentary maximum flow rate of the 15 ℓ drainage is decreased down to 0.25 ℓ/sec when measured at 25 m from the draining point. On the other hand, the 90 ℓ drainage keeps the 0.25 ℓ/sec flow rate even at 300 m from the draining point.

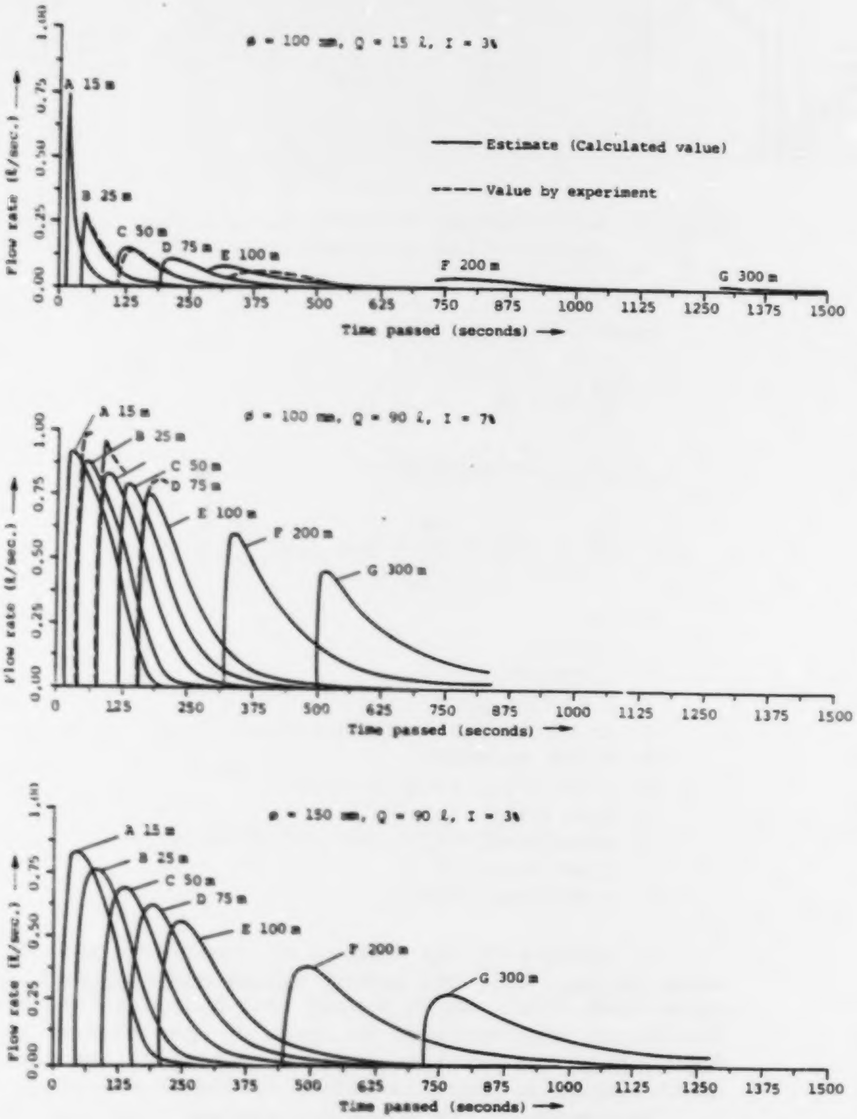


Fig. 11 Changes in flow by time

- o The greater the slope is and the smaller the size of the pipe is, the longer the distance is before the momentary maximum flow rate decreases.

- o Water level and flow velocity have the same tendencies mentioned above.

(2) Feces conveyance function (straight pipe)

The study of hydraulic characteristics tells that water level and flow velocity can be maintained in the branch pipe pretty far down when having the momentary maximum flow rate by the drain from bathtub. In case a bathtub of 90 ℓ is drained to a pipe of 100 mm with a grade of 3%, the flow velocity shall be about 0.5 m/sec. and 80% of the initial water level can be kept downstream up to 100 m. The feces conveyance function by such momentary maximum flow rate is measured by experiment using artificial feces.

(a) Method for experiment

The same equipment used for research of hydraulic characteristics were used. However, pipe of 100 m extension was adopted. With consideration to repeatability, the artificial feces are made of sponge, specifc gravity of 1.5 when wet, and weight 29 g/piece. The shape is imitated to the actual object. Four (4) pieces are discharged at a time by toilet flushing.

The smaller the flow rate is, the more disadvantageous it is for feces conveyance function. The most disadvantageous condition was supposed in the experiment: One house is connected to the branch pipe, four (4) family members living in the house.

The relationship between size of pipe, slope, and feces conveyance function was studied with eight (8) patterns of drainage supposing that the standard type toilet (15 ℓ/flush) or water-saving toilet (ℓ/flush), washing machine (30 ℓ), and bathtub (90 ℓ) are equipped in the house.

(b) Result of experiment

Fig. 12 and Fig. 13 show result of the experiment. The result clarifies the following:

- o As shown in Fig. 12, it is advantageous to use a pipe of 100 mm compared to that of 150 mm under normal drainage condition where 30 ℓ/time of laundry drain can be expected. For example in Pattern 5, feces is conveyed down to 100 m in a pipe of 100 mm while that is conveyance only 30 m in a pipe of 150 mm. Flow velocity and water level have much to do with conveyance of feces. As predicted with hydraulic characteristics of unsteady-state flow, the feces conveyance function is greater when the size of pipe is smaller.

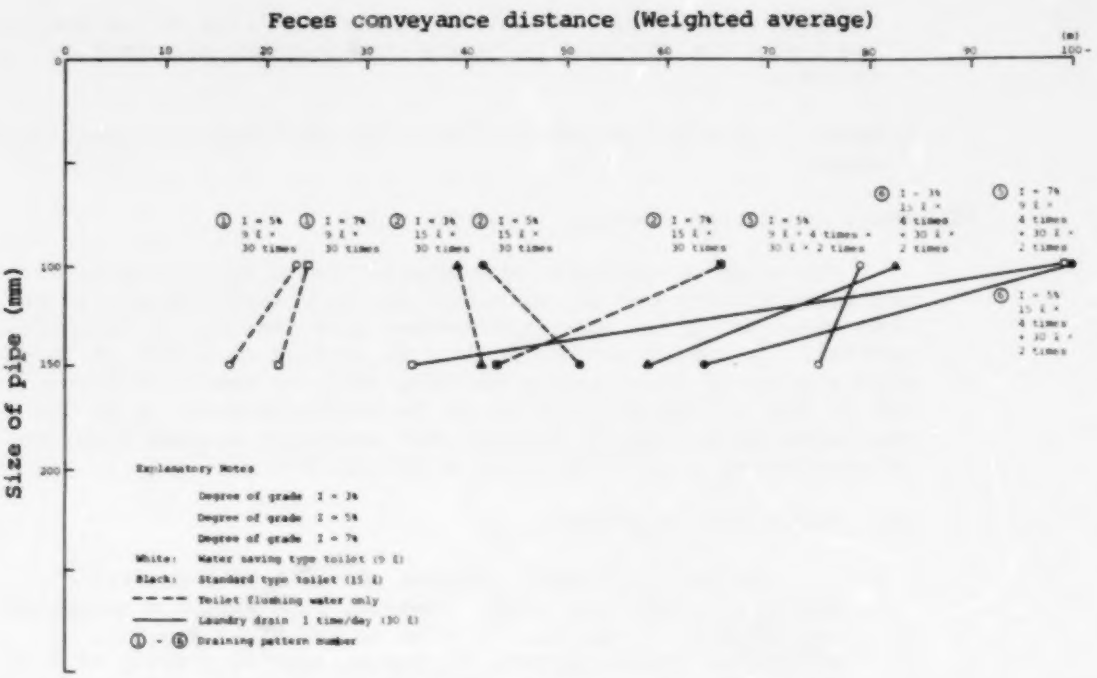


Fig. 12 Relationship between size of egg-shaped pipe and artificial feces conveyance distance

- o As shown in Fig. 13, the slope of 3% is enough for conveying feces 50 - 100 m down in a pipe of 100 mm when laundry drain can be expected every day. If full pipe flow rate of the egg-shaped pipe is calculated using the equation of Manning, the slope of 3% is not appropriate according to the Design Criteria: the equation brings out the flow rate of 0.46 m/sec. However, such slope offers enough feces conveyance function when considering unsteady-state flow depending on the draining patterns.

- o The same experiment was done using the circular pipe. The feces conveyance function of the circular pipe was one half of that of the egg-shaped pipe.

(3) Feces conveyance function (curved pipe)

In order to decrease the number of manholes, the curved pipe allowable under condition of unsteady-state flow was studied from the view of feces conveyance function.

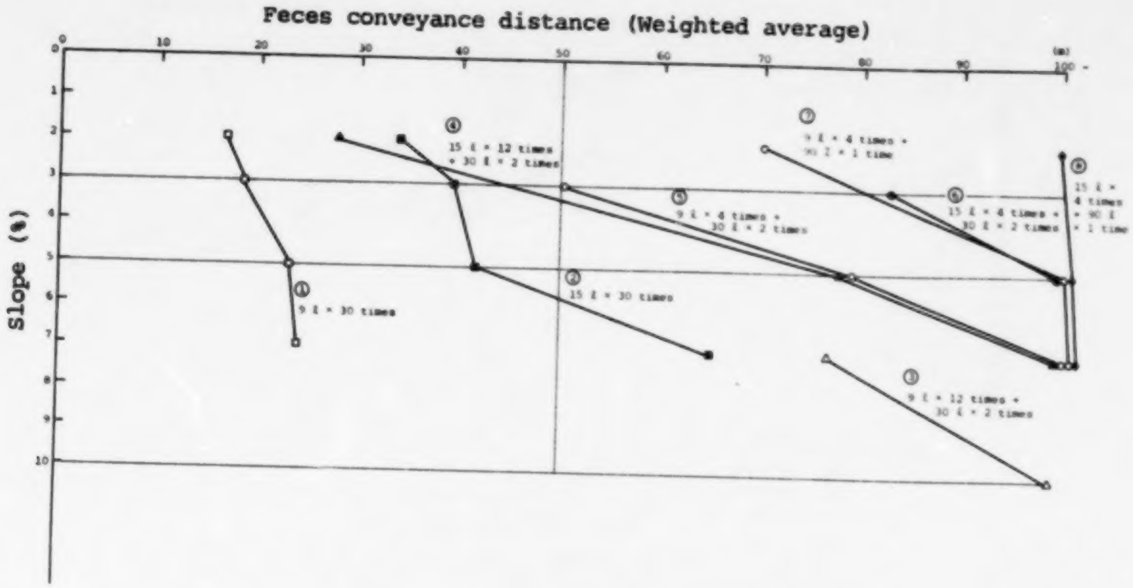


Fig. 13 Relationship between slope of egg-shaped PVC pipe and artificial feces conveyance distance (Diameter: 100 mm)

(a) Method of experiment

As sown in Fig. 14, curved pipes are set at four (4) parts within the distance of 100 m. Experiment was done separately using a piece of 90° bend pipe at each part and using two pieces of 45° elbow pipe at each.

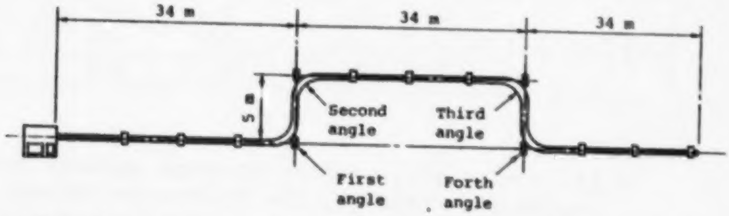


Fig. 14 Experimental setup for curved pipe study

(b) Result of experiment

Fig. 15 shows the relationship between radius of curvature of the curved pipe and feces conveyance distance. Table 6 presents the comparison between conveyance distance in straight pipe and that in curved pipe. The following can be known from the result:

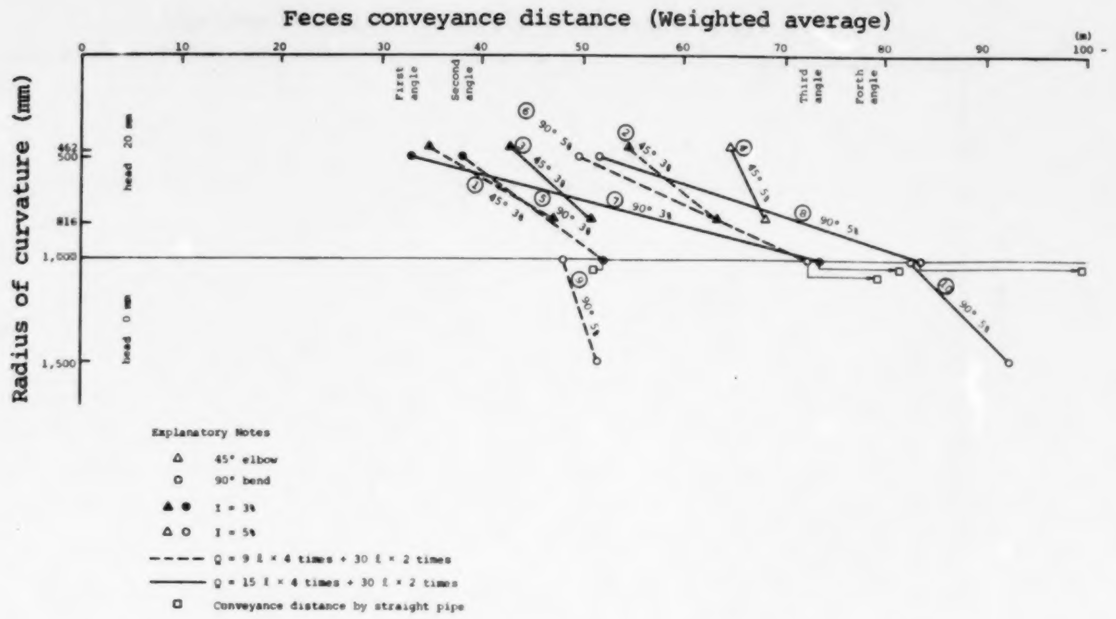


Fig. 15 Relationship between average feces conveyance distance in egg-shaped PVC pipe of 100 mm and radius of curvature

Table 5 Comparison of conveyance distance in curved pipe with 100 cm radius of curvature and straight pipe

Draining pattern	Slope	Feces conveyance distance in straight pipe	Feces conveyance distance in curved pipe	Difference	Difference per bend
9 ℓ x 7 + 30 ℓ x 2	3%	51 m	52 m	- m	- m
	5%	80 m	73 m	7 m	2 m
15 ℓ x 4 + 30 ℓ x 2	3%	83 m	74 m	9 m	2 m
	5%	100 m	84 m	16 m	4 m

o It is more advantageous to adopt greater radius of curvature. However, the difference between conveyance distances of 100 cm radius of curvature and that of 150 cm radius of curvature is smaller than the differences between that of 50 cm radius of curvature and that of 100 cm radius of curvature. Judging from the viewpoint of processing of pipe and work efficiency, the angle pipe with 100 cm radius of curvature seems to be proper.

- As shown by ⑥ and ⑨ in Fig. 15, the feces conveyance function is greatly improved if a head of 20 mm is provided when there is a smaller quantity of sewer compared to that of ⑧ and ⑩.

- There shall be greater difference between the feces conveyance function of the straight pipe and that of the curved pipe as drainage rate and slope become greater. However, the conveyance function per curved pipe with 100 cm radius of curvature does not have much difference compared to that of the straight pipe. (Table 5) In addition, it is quite rare to use several curved pipes in such a short distance as adopted in the experiment. Therefore, it shall be allowed to adopt curved pipe with 100 cm radius of curvature or more and head of 20 mm. This could be considered to have the same function as straight pipe.

(4) Maintainability

From the viewpoint of feces conveyance function, the pipe having size of 100 mm shall be considered to have enough ability. However, there is a possibility to have blockade in a pipe for some reason. Experiment was done to check if any trouble may occur to remove the large substances clogging in the pipe of 100 mm. Clog was artificially made in the experimental equipment containing curved pipes, with artificial feces, paper and cloth diapers, etc. Then the work to remove the clog was done using the Mini-Jet and Flexi-Rodder which are usually used for cleaning sewer. Removal was achieved without any problem.

Employing the pipe of 100 mm instead of the existing minimum size pipe of 200 mm may possibly cause more clogging by large substatnces such as paper diaper. On the other hand, it is easier to know the cause of clogging since a fewer houses are assigned to a branch pipe of the small-scale sewerage. In addition, residents of such communities do not move often and 80% of the agricultural communities achieve road repair and trench scooping in coopration (survey by the National Land Agency). This represents the strong sense of coopratíve body. Residents of such areas easily understand that draining of abnormal substances into sewer causes clogging thereby bringing troubles to themselves. Acordingly, it can be expected that they learn the use of sewerage properly. From this point of view, it also can be said that maintenance of small-scale sewerage sytem with pipes of small size in such communities is rather easier compared to that in city areas.

(5) Maximum flow capacity

The minimum pipe size of 200 mm specified in Design Criteria has usually much room in flow capacity compared to the planned flow rate. Therefore, designing based on steady flow did not cause any problem concerning flow capacity. Now that the size of pipe is 100 mm, it is necessary to design the maximum flow capacity of the branch pipe based on unsteady-state flow, as designing considering sludge conveyance function.

The momentary maximum flow rate of unsteady-state flow shall be greater than that of steady flow. However, the momentary maximum flow rate made by each house shall occur within a very short period of time, and there is little possibility for the momentary maximum flow from each house to coincident.

Therefore, design of branch pipe should be done based on the theory of probability. The study was done about the branch pipe of ø100 mm and the pipe of ø150 mm into which unsteady-state flow may reach in case the branch pipe is short.

① Computation method

The simultaneous drainage probabilistic method used in plumbing facilities was adapted for the study mentioned above. The basic equations are described below.

Where w (ℓ) represents the volume of water drained from one unit at a time, and q (ℓ/sec) represents the flow rate of w flowing the pipe, then the time for draining t (sec) is expressed as follows:

$$t = \frac{w}{q}$$

The time ratio (P) shared by w from a certain time T to T + T_0, that is, during the time duration of T_0, can be expressed by the following equation:

$$P = \frac{t}{T_0} = \frac{w}{T_0 \times q}$$

When w is drained from n plumling fixture units within the time duration T_0, then the time ratio (the average number of simultaneously draining units) shall be expressed as follows:

$$\mu = n \cdot p = \frac{n \times w}{T_0 \times q}$$

In the case n is extremely big and P is small, the probability to have simultaneous draining is known to closely be approximated by the Poisson distribution. The probability to have simultaneous draining from r units shall be expressed as follows:

$$P\ (r) = \frac{\mu^r \cdot e^{-\mu}}{r!}$$

From this equation, the total probability (α) to have simultaneous draining from more than m plumbing fixture units can be expressed as follows:

$$\alpha = \sum_{r=m}^{\infty} P\ (r) = \sum_{r=m}^{\infty} \frac{\mu^r \cdot e^{-\mu}}{r!}$$

Then the maximum simultaneous drainage rate Q (ℓ/sec) can be obtained by the following equation when the average number of simultaneously draining units is μ, and the maximum number of simultaneously draining units for α is m.

$$Q = m \cdot q$$

Sanitary sewage from each home includes bathtub drain, laundry drain, kitchen drain, toilet flushing drain, etc. Some or all of these possibly occur simultaneously as an indenpendent events. Sanitary sewage from each drainage unit shall be considered as an indenpendent events with consideration to safety.

P (A) represents the simultaneous draining probability from a certain number or more of a certain kind of plumbing fixture unit. P (A_1, A_2, ..., A_i) represents the probability that each event of i kinds of plumbing fixture unit occur simultaneously. P (A_1, A_2, ... A_i) is expressed as follows:

$$P\ (A_1,\ A_2\ \ldots\ A_i) = P\ (A_1) \times P\ (A_2) \times \ldots \times P\ (A_i)$$

And when Q_1, Q_2, ... Q_i represent the maximum simultaneous drainage rate of each kind of unit, then the total maximum simultaneous drainage rate (Q) that is caused in case these drains are discharged simultaneously is expressed as follows:

$$Q = Q_1 + Q_2 + \ldots + Q_i$$

The number of units, that is, the number of houses can be estimated by comparing the hydraulic flow capacity of pipe itself with the maximum simultaneous drainage rate under a certain probability obtained from these basic equations.

② Condtions for computation

The following conditions are defined for computation.

(a) Plumbing fixtures concidered and volume of water per discharge

To know the maximum simultaneous drainage rate, simultaneous drainage rates all of the drainage units should be obtained and totaled. For simplification of computation, however, the units whose drainage rate is small were neglected. That is, computation of the maximum simultaneous drainage rate was done using bathtub drain and laundry drain. The quantity of drainage per time from each drain was supposed 170 ℓ/time and 40 ℓ/time based on the actual measurement.

(b) Draining time period

For study of draining time period, drainage patterns of three (3) communities were researched. The research tells that the time period for bathtub drain and laundry drain is generally divided into two: in the morning and/or in the afternoon. Laundry drain occurs more in the morning, while bathtub drain appears slightly more in the afternoon than in the morning. Such drains are concentrated within eight (8) hours a day.

The draining time period may incline to occur in either of the morning or afternoon. Therefore, the draining time period of four (4) hours was adopted for computation. In addition, the draining time period of two (2) hours was used for computation because there was occasionally a tendency for drainage to occur concentrated in two (2) hours. These two cases (2 hours, 4 hours) are used for computation.

(c) Pipe arrangement and density of houses

The area to which the small-scale sewerage system is applied has population of 49 heads/ha. or less. So, the density of houses was supposed to be 10 houses/ha. based on the standard. The model was made with houses located under such condition, with 15 m sewer extension from the draining point of each house.

(d) Momentary maximum flow rate

The momentary maximum flow rate of bathtub drain and laundry drain measured at 15 m downstream from the draining point are as follows. They are obtained via the simulation described in Section 3.2.1.

Bathtub drain (170 ℓ): q = 1.07 (ℓ/sec)
Laundry drain (40 ℓ) : q = 0.44 (ℓ/sec)

The sewer extension from each house to each designed point practically differs. However, the 15 m sewer extension, which is on the safer side, was adopted for simplification.

(e) Excess probability

To set some computing cases, the excess probability based on the Poisson's distribution was adopted as follows: excess probabilities from 1/100 (once per three months) to 1/10000 (once per three yeras) were applied for each drain.

(f) Hydraulic flow capacity of pipe

When the maximum flow rate of sewer is set based on a certain excess probability, there is possibility a flow exceeds the flow rate set with the probability. Such case does not necessarily cause an overflow at manhole and/or catch basin, because the flow shall be pressurized and go downstream in most of the case.

The flow rate considered herein is a momentary flow rate. Therefore, pressurized water flow should be allowed when having a flow rate over a certain probability. The hydraulic flow capacity of pipe was computed separately for full pipe flow rate and pressurized flow rate to be compared with the maximum simultaneous drainage rate.

③ Results

Table 6 shows the number of houses acceptable for drainage depending on the excess probability, using a pipe of ø100 mm and ø150 mm were used with a grade of 3%. In case of 100 mm and the draining time period of four (4) hours with excess probability of 1/1,000 for full pipe flow, for example, then the number of houses acceptable is 50. In case of excess probability of 1/10,000 for pressurized flow, then the number of houses acceptable is 42. This capacity can be said enough practically when considering the standard density of houses (10 houses/ha.).

In case a pipe of 150 mm is used and the draining time period is four (4) hours with excess probability of 1/10,000, then 135 houses can be accepted. The model arrangement of houses requires sewer extension over 1,000 m, thereby greatly decreasing the momentary maximum flow rate genrated by drain from each house. This means that such design is further on the safe side.

As a result of the above-mentioned studies and researches, the pipe of 100 mm can be used with a minimum slope of 3% for small-scale sewerage system so as to decrease the construction cost without causing any problem in large substances conveyance capacity and maximum flow capacity.

Table 6 List of maximum flow capacity (number of houses)

Total drain excess probability				1/100			1/1000				1/10000				
Bathtub drain excess probability				1/1	1/10	1/100	1/1	1/10	1/100	1/1000	1/1	1/10	1/100	1/1000	1/10000
Laundry drain excess probability				1/100	1/10	1/1	1/1000	1/100	1/10	1/1	1/10000	1/1000	1/100	1/10	1/1
Type of pipe diameter (Degree of grade)	Average draining time period	Item	State of flow												
Egg-shaped PVC pipe ø100 mm 3‰	7200 sec. (2 hr.)	Number of houses acceptable for draining	Full-pipe flow	70	44	45	50	33	26	* 25	38	27	20	16	15
			Pressurized flow	80	51	54	59	41	34	35	48	35	29	* 25	26
	14400 sec. (4 hr.)	Number of houses acceptable for draining	Full-pipe flow	140	91	90	100	68	54	* 50	76	53	39	33	30
			Pressurized flow	150	100	100	110	76	62	60	86	61	48	43	* 42
Egg-shaped PVC pipe ø150 mm 3‰	7200 sec. (2 hr.)	Number of houses acceptable for draining	Full-pipe flow	215	115	150	175	95	* 83	105	145	84	69	63	79
			Pressurized flow	225	125	160	185	105	93	115	160	93	78	* 72	91
	14400 sec. (4 hr.)	Number of houses acceptable for draining	Full-pipe flow	440	235	305	360	195	* 170	210	300	170	140	125	160
			Pressurized flow	450	240	310	370	200	180	220	310	175	150	* 135	175

3.2.2 Simplified manhole

The inside diameter of existing manhole is 90 cm at the minimum for maintenance works of sewer to be done in a manhole. Maintenance works of small diameter pipe with small earthcovering is considered to be done from the ground.

Provided that man would not enter manhole, the diameter of a manhole can be made smaller unless it does not cause any trouble to use flushing equipment therein. Then, construction cost of manhole can be saved.

Based on this theory, simplified manhole has been devised. The peculiarity of this simplified manhole is that the frame of cover and the top slab are not directly attached to the vertical wall thereby conducting load on cover directly to the ground. Such structure decreases the load to be conducted to the base slab, avoids settlement of sewer and reduces stress to the vertical wall. Ready-made product shall be used as an invert at the bottom and installed by bonding. To connect a sewer to the vertical wall, a combined saddle is to be used for maintainability.

Strength test and study of maintainability were achieved using a prototype. The loading test by the normal wheel load was applied to measure the stress at each part and settlement rate. The result tells that the stress generated at each part in greatly under the strength of the part, that is, each part can bear the wheel load. It is also proved that the settlement caused by full load does not cause any practical problems. As for maintainability, pipe check (inside) by a T.V. camera and cleaning by Mini Jet were done using the prototype. It was proved that the manhole of 300 mm diameter does not give any troubles to the works using the T.V. camera and Mini Jet.

It was also proved that small manholes have the following advantages in construction compared to the existing standard manhole:

(a) Requires the same excavation width with a pipe thereby decreasing earth works.

(b) Requires a very short time for construction thanks to unnecessity of concrete curing.

(c) The construction work is very simple thanks to employment of a ready-made product as an invert.

(d) Assemble and installation works can be done by man power thanks to the light material.

(e) There is a clearance between the vertical wall and the top slab, which allows delicate adjustment in setting the manhole cover height.

3.3 Development of Treatment Facility

3.3.1 Cost analysis

As known from experiences, the construction cost for sewerage treatment facility becomes rather higher considering the scale when the treatment facility is smaller. To hold the construction cost for small-scale sewerage treatment facility, the factors rising the cost have been studied.

The study does not include the control building and sludge building which have been clarified so far.

Three (3) ranks of scales were provided for comparison: 1,000 m^3/day, 500 m^3/day, and 100 m^3/day. Three (3) kinds of treatment methods were used: conventional activated sludge process, extended aeration process, and oxidation ditch process. As for computation method of cost, estimations done by several sewerage facility makers were referred. Basic factors and specifications of main facilities were presented to them. As one of basic conditions, it was ruled that a grid chamber and a private power plant should not be established.

Fig. 16 shows construction cost per design flow classified based on the scale and the treatment method. This figure shows that the smaller the scale becomes, the higher the construction cost per design flow becomes. The ratio of construction cost per design flow among 1,000 m^3/day and 500 m^3/day is 1:1.3 - 1.2. Such ratio among facilities processing 500 m^3/day and 100 m^3/day is greater, that is 1:2.0 - 2.1. Every cost item requires higher unit cost when the scale becomes smaller. Especially the facility processing 100 m^3/day has greater rise in ratio. As for oxidation ditch process, the ratio between the construction cost per design flow among facilities processing 500 m^3/day and 100 m^3/day is 1:1.4 in construction facility and 1:2.6 in machinery equipment. The rate of machinery equipment in the facility processing 500 m^3/day is 40%, while that in the facility processing 100 m^3/day is 51%.

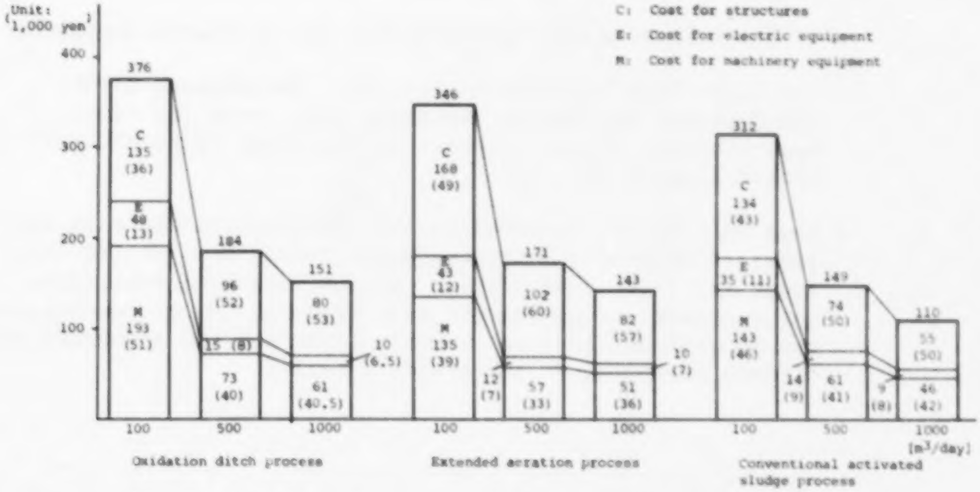
The reason for the rise in construction cost of machinery and electronic equipment per design flow for the facility processing 500 m^3/day or less is due to the limit in minimizing scale of each equipment. This means that the capacity of facility does not become smaller in proportion to the scale of wastewater treatment flow.

As for aeration unit, the electric motor output per 100 m^3/day was compared among facilities processing 100 m^3/day, 500 m^3/day, and 1,000 m^3/day. It resulted in 1.5 kW, 1.1 kW, and 1.1 kW at each facility. The facility processing 100 m^3/day requires the highest considering the scale. The collector of settling tank is operated with the same power used for all the facilities processing 100 m^3/day, 500 m^3/day, and 1,000 m^3/day. Automatic screen of the same size is used for all the facilities. This means that the construction cost per design flow of the facilities processing 100 m^3/day is ten (10) times of that of the facility processing 1,000 m^3/day. Main pumps provided for the facility processing 1,000 m^3/day are 2 units (5.5 kW) and 1 spare unit. Those for the facility processing 500 m^3/day are 1 unit (5.5 kW) and 1 spare unit. Accordingly, the facility processing

500 m^3/day cost much requiring more spare unit considering the scale.

The ratio of principle items in the machinery equipment was studied. Fig. 17 shows that of the oxidation ditch process handling 500 m^3/day. The aeration unit, control unit, and sludge collector hold a grat portion. As a result, it can be said that improvement of these equipments serves effectively to down the cost.

The cost analysis has clearified that it is necessary to develop the technology to make construction facility, machinery equipment, and electric equipment small scaled. Especially the machinery equipment need be small scaled. Standardization of facilities and study for practical use of sludge collector and sludge dewatering bag have also been studied. They are described below.



Note: Each value in ( ) indicates the percentage of the each item.

Fig. 16 Construction cost per design flow of 1 m^3/day (prices as of 1982)

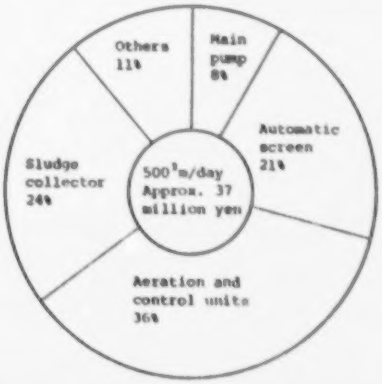


Fig. 17 Breakdown of machinery cost

3.3.2 Standardization of facilities

The small-scale sewerage treatment plant has greater scale demerit, and a great number of small-scale sewerage treatment plants are expected to be constructed in future. The scale demerit in the construction cost shall be decreased by standardizing facilities thereby allowing mass production of parts. In addition, mass production possibly allows simplification of construction, maintenance, and reduction of construction period. Stadardization also omits designing work which has been done individually for each facility. At the same time, design cost and office overheads can be cut. Standardization is now in the work to consider mass product at factories. The outline is described below.

(1) Basic policy

The following were provided for one of standardizations:

- o The basic flow is shown in Fig. 18. The objects to be standardized are screen, aeration tank, settling tank, hypochlorite contact channel, and thickner (part enclosed by broken line in Fig. 18).
- o Pump well is not included for standardization since it may not be required depending on geographical conditions, or its inlet sewer level and water head differ. Sludge treatment facilities is also not included for standardization since sludge processing/disposal can be centralized from srveral plants and the treatment method differs depending on the disposal method.

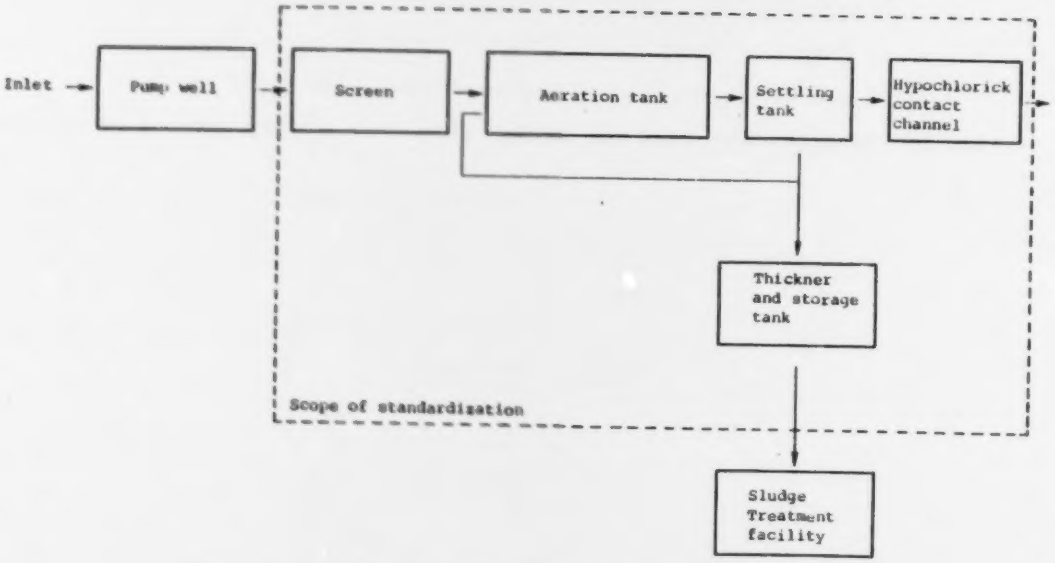

Inlet
Pump well
Screen
Aeration tank
Settling tank
Hypochlorick contact channel
Thickner and storage tank
Scope of standardization
Sludge Treatment facility

Fig. 18 Basic flow and scope of standardization

- Existing treatment plants have two (2) series or more to secure treatment ability. In this standardization, however, the teratment series shall be one (1) from the economical view-point. Accordingly, it is required to make it possible to achieve check and/or repair without stopping treatment.
- Small-scale wastewater treatment plant tends to hold a great portion of personnel expense among maintenance cost. To reduce the personnel expense, the facility shall be designed to require patrols about two (2) times a week instead of hiring full-time operators in principle.

Based on this theory, standardization of oxidation ditch process has been tried at six (6) ranks (300 m^3/day - 1,200 m^3/day) which are within the range of scale set in the Small Facility Design Manual.

(2) Structure

When making a standard design, the structure of each facility shall be based on the following theories:

- Principally, civil engineering works shall not require landslide protection wall. Facility load is small thereby employing the spread foundation as standard foundation type. Foundation type is required to be designed somehow else in case pile foundation.
- The aeration tank and the final settling tank shall be the integrated concentric circle type which decreases the number of wall.
- Base slab level of the aeration tank and that of the final settling tank shall be the same, that is, the base slab of the final settling tank shall be flat without any taper. This will make the construction easy and reduce cost. Sludge collection function of a settling tank with a flat base slab is described later.
- Field assembly using precast concrete parts or employment of ready-made concrete products for buildings are to be promoted so as to encourage mass production and commercialization.

(3) Selection of equipment

The points to be considered for selection of equipment are shown as follows.

- Automatic equipment will maintenance free should be positively adopted since it is operated by patrol system in principle.
- High technology of the operation and maintenance shall not be needed since it is difficult to secure patrol man with such

technology.

- For easy storage of spare parts and replacement, products on the market shall be used as far as possible.
- The structure need be designed to allow check, repair, and replacement without stopping treatment, since only one (1) series exists.

3.3.3 Simplified sludge collector

Equipment of treatment plant has also been developed considering the standardization theory. The simplified sludge collector devised during the development, is outlined below.

The sludge collector used in the existing small-scale treatment plant has fixed rake and center pole. This requires to temporarily empty settling tank for check and/or repair of the sludge collector. Accordingly, they have two (2) serieses. To improve demerits of such sludge collector, a simplified sludge collector has been designed which does not need to empty settling tank for check and/or repair.

Fig. 19 shows its concept. Th rake is hanged by a chain from the arm which turns around the center pole by means of drive unit. Sludge is collected by the turning rake. The rake, hanged by a chain, can be accessed for repair or replacement easily by hauling the chain, without emptying the tank. In addition, the weight of material shall be decreased since the parts are not fixed, thereby causing a cost reduction.

Using this type of collector, experiment was done to confirm sludge collection function. The base slab of the tank is flat considering economical advantage and easy construction. The rake is curved so as to collect sludge stabilizedly.

The experiment proved that the sludge collector of such structure does not have any problem in its operation and ability.

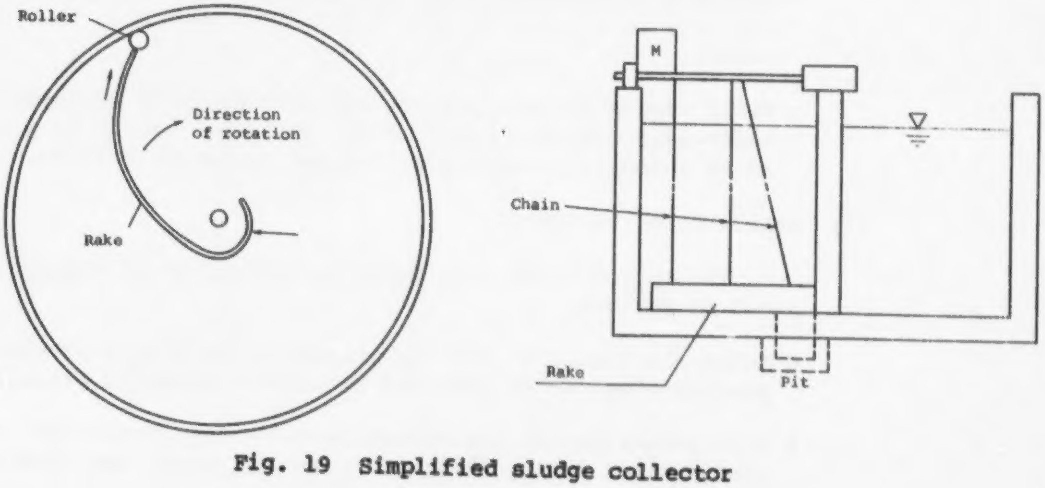


Fig. 19 Simplified sludge collector

3.3.4 Sludge dewatering bag

Though it was not mentioned in the section of cost analysis, it is known sludge treatment facility has a great scale demerit. For example, expenses per sludge volume for dewatering by machine are higher in smaller-scale. This is because the dephdrator of small-scale plant has sometimes greater capacity than that is required since the dehydrator has not been developed for small-scale plant. In addition, experts in machinery and electrical engineering are required for operation and management. The sand bed process has an advantage of cost reduction while requiring pretty much labor for removing dried sludge.

In the "Small Facility Design Manual", centralized sludge treatment and disposal system is recommended to save energy and increase economical advantages for small-scale sewerage treatment plant. When such centralized system is adopted, then the scale merit of sludge treatment facility shall be increased as well as opportunity to secure experts in machinery and electrical engineering.

On the contrary, it requires each community to promote sewerage works at the same level, and there are other practical problems. A simple method for dewatering is desired at least for present. From this point of view, the Japan Sewerage Works Agency has studied practicability of sludge dewatering bag.

Fig. 20 shows the concept of the sludge dewatering bag. Excess sludge, after mixed with polymer, is put into the bag made of Tetoron or rayon to be left for a certain period for dewatering and drying. The cloth of the bag plays the filtering role of sand of the sand bed. The sludge shall be disposed as it is in bags when dewatering and drying is complete.

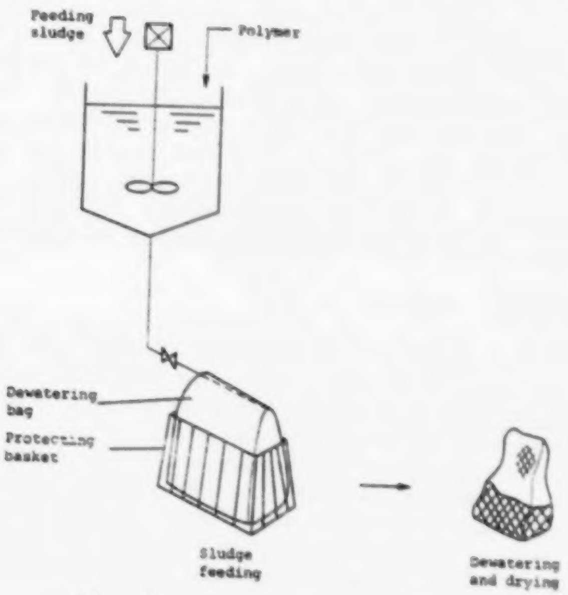


Fig. 20 Sludge dewatering bag

The construction cost is extremely low since hardly any equipment is required as mentioned above. The required site shall also be smaller compared to that is required for sand bed process. Mechanical and electrical engineer are not needed either because of such simple equipment.

The excess sludge from the long-period aeration process was used in experiment to check the effect of sludge dewatering bag. It resulted that the sludge was dewatered to 70% moisture content after four (4) weeks drying. Provided that a bag of W 100 cm x H 75 cm can treat sludge containing about 5 kg of dry solid, then forty (40) bags are required in a week at the treatment plant of 300 m^3/day. Considering the labor for feeding sludge and carrying bags, the sludge dewatering bag process is adaptable only at a treatment plant of extremely small-scale. It can be said the sludge dewatering bag process is easy to be employed as a temporary method until centralized processing/disposal system for a wide area is prepared.

3.4 Theme for Future

(1) Collection facility

It has been proved that the pipe of 100 mm with a grade of 3% can be used when unsteady-state flow in the branch pipe is counted. However, substitute sludge made of sponge was used and the draining patterns were limited in the experiments. Various objects such as toilet paper, vegetable scraps, cloth, vinyl as well as feces are disposed into the practical sewer. In addition, there are other conditions which were difficult to be checked by experiments (such as influences by catch basins or house connection pipes). Proof tests using practical facilities are indispensable for practical use. Practical laying tests have already been started in a certain line. In order to prove practicality under various conditions, many practical laying tests are being planned. After such checks and tests, a munual to help easy designing of sewer shall be prepared. As for simplified manholes, they are already under manufacturing since no trouble was found in experiment. Proof tests, however, shall be performed at practical facilities with simplified manholes so that they can be diffused.

(2) Treatment facility

It has approximately been proved that the simplified sludge collector and sludge dewatering bags can be practically used. Whe inconveniences found in experiments are improved, proof tests need be achieved at practical plants.

Through standardization works for oxidation ditch process, the basic theory of standardization of small-scale sewerage treatment plant could have been clarified. Technology for small-scale equipment (such as simplified sludge collector) need be promoted at each facility so as to improve maintainability and to decrease construction cost by standardization.

In preparation for increasing needs for small-scale sewerage system, establishement of technology for standardization and mass production need be promoted continuously as well as improving conditions for such producton and marketing.

EFFLUENT REUSE IN AN URBAN RENEWAL DISTRICT IN TOKYO

The work described in this paper was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

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1. INTRODUCTION

The demand for water is constantly increasing along with the massive concentration of population in Tokyo, the city's economic development and modernization of city life.

Water supply in Tokyo is expected with the maximum daily supply reaching an estimated 6.89 million m^3 in fiscal 1985, and 7.48 million m^3 in fiscal 1990.

Tokyo's water works have now water sources, available to use, amount to 5.63 million m^3. So development of water resources, such as reservoir, is requested to implement.

Meanwhile, the construction of new reservoirs is hampered, because of the scarcity of exploitable sites, and the need for environmental protections.

To ensure effective use of limited water resources, Tokyo Metropolitan government has taken the following measures.

2. AIMING AT FURTHER RELIABLE WATER SUPPLY

(1) Water works expansion project

Tokyo's water works takes most of its water from the rivers of three water systems. Those are shown in Fig.-1.

a) Tama River water system

b) Tone River water system

c) Sagami River water system

In order to meet the growing water demand, the Fourth Tone Water Works Expansion Project was launched in Fiscal 1972.

The project is designed to (1) construct Misato purification plant to expand installed capacity, (2) strengthen the pooling function of purification plants through construction of pumping stations and communication pipes, and (3) carry out emergency measures to ensure effective raw water utilization through construction of a second intake facilities at the Ogochi dam, an intake weir at ozaku and conveyance pipes.
To cope with this project, new reservoirs for raw water will be needed.

(2) Leak prevention

Priority is given to leak prevention from the view point of effective utilization of available water resources.
The leaks that occurred in fiscal 1984 accounted for an estimated 14.7 percent of total supplied volume.
Service pipes account for more than 90 percent of the leak repair. To drastically improve the pipe structure and material, stainless steel pipes were used biginning in May 1980.

(3) Promotion of Demand Restricting Measures

Water is essential for the preservation of city life.
In view of the current tight condition of water supply, the rational use of water will be expected.

In January 1973, the Bureau announced "Measures to Restrict Water Demand". The following actions have since been taken for this purpose.

a) Campaign for water conservation such as pamphlets and poster

b) Development and use of conservation-Type Equipments such as faucet, washing machine, flush toilet.

c) Water conservation through education of children. Study materials are distributed to all fourth-graders of primary schools and first-year students of junior high school in the water supply areas in order to propagate a correct knowledge of water works through education.

(4) Recycling of used water

Because of the difficulties in evacuation of residents, and development of adjacent area, construction plan of new reservoirs have been delayed. Therefore, the unit cost of water resources development will be rising.

On the other hand, diffusion rates of sewerage for 23 wards of Tokyo, reached to 82 percent in fiscal 1984, and the volume of sewage treated amounts to nearly 6 million m^3 per day.

So, recycling of used water will be very important not only for solving the shortage of raw water, but also preventing water pollution of watersheds of source rivers.

And recent tendency has been for both city water rates and sewage charges to increase greatly for large consumers.

In Tokyo, total charges can amount to more than ¥600/m^3.
For this reason, the augment for waste water reuse becomes very persuasive, not with standing the fact that the cost of reuse has also increased.

3. PRESENT SITUATION OF TREATED EFFLUENT REUSE

(1) Reuse in industrial water supply

The first attempt to reuse treated municipal sewage for industrial water started in 1951, when treated effluent at Mikawashima sewage treatment plant in Tokyo was sand-filtered and supplied as a trial to a cardboard manufacturing factory.

At the period 1960s, many factories served underground water for industrial use.

Of the total number of factories in the 23 special wards, about one-half are located in the industrial areas of Koto and Johoku. Enormous quantities of underground water were pumped up in these areas until restrictions were imposed.

The heavy intake of underground water has caused a conspicuous ground subsidence in these industrial areas. At one place the ground level has dropped more than 4m during its past 60 years.

The metropolitan government supplied substitute industrial water to the areas in 1965 where restrictions are placed on the use of underground water to prevent the sinking of the ground.
Alternative industrial water is supplied through two channels. One is the industrial water works in the Koto area. The other is in the Johoku area.

Thanks to these projects the ground sinking in the Koto and Johoku districts has slowed markedly since the 1970s. At present there is little evidence of subsidence.
Thus the purpose of industrial water works-prevention of ground subsidence - has been achieved.

The source of Koto industrial water works is sewage which is treated by the activated sludge process at the Mikawashima sewage treatment plant. The treated sewage then undergoes sedimentation, filtration and sterilization at the Minami-senju purification plant before being distributed to individual factories.
On the other hand, Johoku industrial water works receives raw water from Tone, and Tamagawa rivers. Surface water from the Tone is led to Misono purification plant, where it is treated through sedimentation. Meanwhile, surface water from the lower reaches of the Tamagawa is treated through sedimentation and filtration at the Tamagawa purification plant, then send to Misono purification plant. At Misono, treated water from these two sources is mixed at the ratio of 3 for Tone and 1 for Tamagawa.

In the case of users downflow of Kohoku purification plant, an average volume of 7000 m^3 per day (normal time) from the Kohoku purification plant is added to the mixed industrial water described above.

At Kohoku, industrial water from Minami-senju, which originates from treated sewage, is further treated with activated carbon to improve quality. This water makes up for the shortages of river water.

The demand for industrial water is due to the Japanese economy's shift to a period of slower growth, evacuation of industrial facilities following imposition of stringent environmental restrictions, and internal use of reclaimed water.

Given these limiting factors, demand has been diminishing.

The bureau plans to use part of the industrial water for other miscellaneous purposes, such as cleaning at factories, scouring, and cooling at sewage treatment plant, car washing at taxi companies and toilet flushing at housing complex. Table-1 shows the proportion of usage of industrial water supply.

(2) Reuse in sewage treatment plant

In 23 special wards of Tokyo, 10 treatment plants are in operation. In these facilities, 250,000 m^3 of water are consumed, for cooling, washing every day. And among them, about 230,000 m^3 are supplied by treated sewage after being filtrated.

(3) Reuse for restoration of waterways

Citizens in Tokyo have a great concern for improvement of urban facilities such as parks and waterfronts.

The Tokyo metropolitan government commenced the project for the restration of waterways to improve the waterside environment.

Tamagawa waterway and its diverted waterways which had supplied water for livelihood and irrigation for over 300 years, were stopped supplying water in 1978, because of no need for irrigation.

However, Nobidome watersupply, which is a typical diversion of Tamagawa watersupply, was restored in 1984, which tertiary treated water began to flow into the waterway from the Tamagawa Joryu treatment plant at the rate of 20,000 m^3 per day. In near future other waterways will plan to be restored.

(4) Reuse for others

In addition, tertiary-treated water is also used for washing Shinkansen (bullet train) carriage of the Japan National Railways.

4. SEWAGE RECYCLING MODEL IN SHINJUKU URBAN SUBCENTER

(1) Outline of Shinjuku urban subcenter

Business firms, by nature, tend to gather at a certain place because they can benefit from mutual contract. Particularly since Tokyo is the capital city of Japan, it has a great number of central government offices and advanced research institutions which is a major reason for the concentration of business in the capital.

Development of new traffic and communication systems, has served to heighten the importance of central management of functions in business firms encouraging independence of such functions.

In order to cope with the situation, The Tokyo metropolitan government carried out urban renewal project in Shinjuku, where Yodobashi purification plant was located.

Yodobashi purification plant is the first modern waterworks, put into service in our city, in 1898, using slow sand filtration process. To meet with the rapid demand of water, Higashi murayama purification plant, equipped with rapid sand filtration, went into operation in 1960. And the sites, involving Yodobashi purification plant, were planned to be used for urban renewal.

Among the planned 96 hectares, 50 hectares are now implementing the urban renewal project. Since 1960, 12 high rise building for hotel, and business offices, are constructed.

And the population working in this district will be expected to 300,000 in near future.

(2) The aims of the sewage recycling project

Tokyo metropolitan government set forth the administrative guidance for providing installation of recycling facilities at the time of construction of large building or a housing complex.

In this systems, individual buildings have their own sewage treatment facilities and the treated water is reused for flushing toilets within the buildings. (Table-2)

After the announcement of establishing of a water-saving society, recycling systems in individual buildings are tending to spread gradually. (Table-3)

And recycling in the urban development area began operation to supply reclaimed water to three building by Marunouchi Central Heating Co. in 1976.

But recycling of used water in large scale has not been established yet.

Therefore, since 1978, suffering from the water shortage, Tokyo metropolitan government intended to implement sewage recycling project as a model in the Shinjuku urban subcenter in order to clear the problems in employing water recycling system in large scale.

And Tokyo metropolitan government set up a guideline for advising citizens for recycling of water in 1984.

On the other hand, water recycle systems in district wide works and individual building within the public sewerage area have a great effect in reducing sewage discharge.

Tokyo's first sewer system, "Kanda sewage works" laid down brick sewer pipes in 1884. So in some parts of public sewered area, the capacities of sewerage facilities became shortage owing to the rapid urbanization and changing in life style.

Therefore, in this point of view, introducing large scale recycling systems to this Shinjuku urban renewal district will be very effective procedures for reducing the capacity of trunk sewer in this area.

(3) Outline of Shinjuku sewage recycling project

In 1984, sewerage bureau with the aid of the Ministry of construction, constructed dual supply system in Shinjuku urban subcenter.

Among the 12 high rise buildings which were already in use, 9 buildings were prepared for dual supply systems following to the advice of the bureau.

a) Projected dual supply area

Projected area was decided on the 50 ha of urban renewal district where the water consumption will be large and reasonable usage of water will be expected. (Fig.-2)
The execution plan was devided into two stages.
First stage is to supply sand-filtrated effluent to a buildings in use, from Ochiai treatment plant located near to subcenter. Second stage is to supply reclaimed water from the Recycling center located at Shinjuku Kokusai building to the whole consumers in the projected area.

b) Design capacity

Planned capacity is 4000 m^3/d for first stage, and 8000 m^3/d for second stage.
Construction programme was devided into two stages owing to the effectiveness of supplying systems.
Capacity for first stage was planned according to the information submitted by the owner of buildings.

The whole capacity for the second stage was calculated using the water-consumptions by unit floor area ($m^3/m^2/d$) and accumulated office floor space (m^2) which will be designed according to the city planning law, and building code. (Table-4)
The capacity for the existing buildings is the same as that of first stage.

c) Supply systems

At first stage, wide area supply system was applied because of the small demand for used water.
In future, accompanied with the development of this district, more waste water reuse will be expected. Therefore, at the second stage, district wide supply system will be adopted, in order to cover the insufficient capacity of existing sewerage pipes.

d) Usage of reclaimed water

The usage of reclaided water have to be decided reffering acceptability of citizens and safety. Furthermore much volume of reuse should be expected.
Shared ratio for flushing toilet, investigated in the office of the Shinjuku urban subcenter, such as Shinjuku NS building, and Shinjuku Center building, were 43 ∿ 50 percentage. (Table-5)
Meanwhile, the ratio for car washing and cleansing were very small. And in this district, central heating and cooling service will be planned to introduce in future.
On the other hand, avoidance of use in direct contact with human bodies may reduce the possibility of sanitary problems, and also the consumer may have less feelings of resistance from the psychological viewpoints.
At present, Ministry of construction and Tokyo Metropolitan government suggested a guideline to use reclaimed water for flushing of toilet.
In this sewage recycling project, reclaimed water was planned to be used for flushing toilet.

e) Water quality for supply and treatment facilities

The water quality for supply should be satisfied the following points.

① To secure safety for the health of consumers.

② The color and the odor of supplied water is not unpleasant for consumers.

③ Dual water supply systems should not be interrupted by scalings and corrosions.

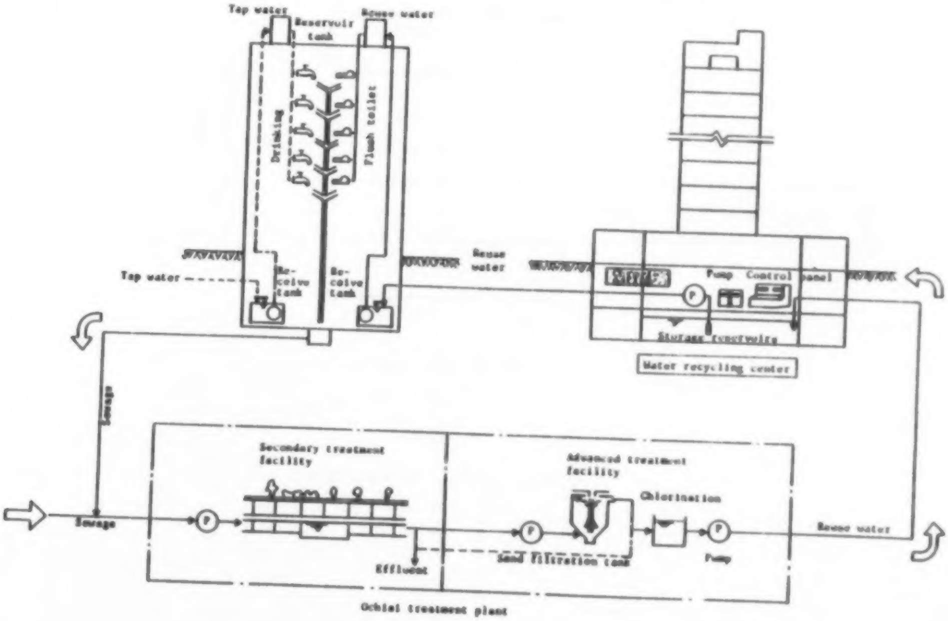
In these point of view, some water quality criteria were proposed. Table-6 shows that for industrial water. Refeering the "Sewage Effluent Reuse Guideline" published by sewage department of Ministry of construction, the water quality for supply was decided as following table.

Items	Criteria value
e col.	less than 10 N/mℓ
residual chlorine	Maintained
Color	Not unpleasant
Odour	Not unpleasant
PH	5.8 ∿ 8.6

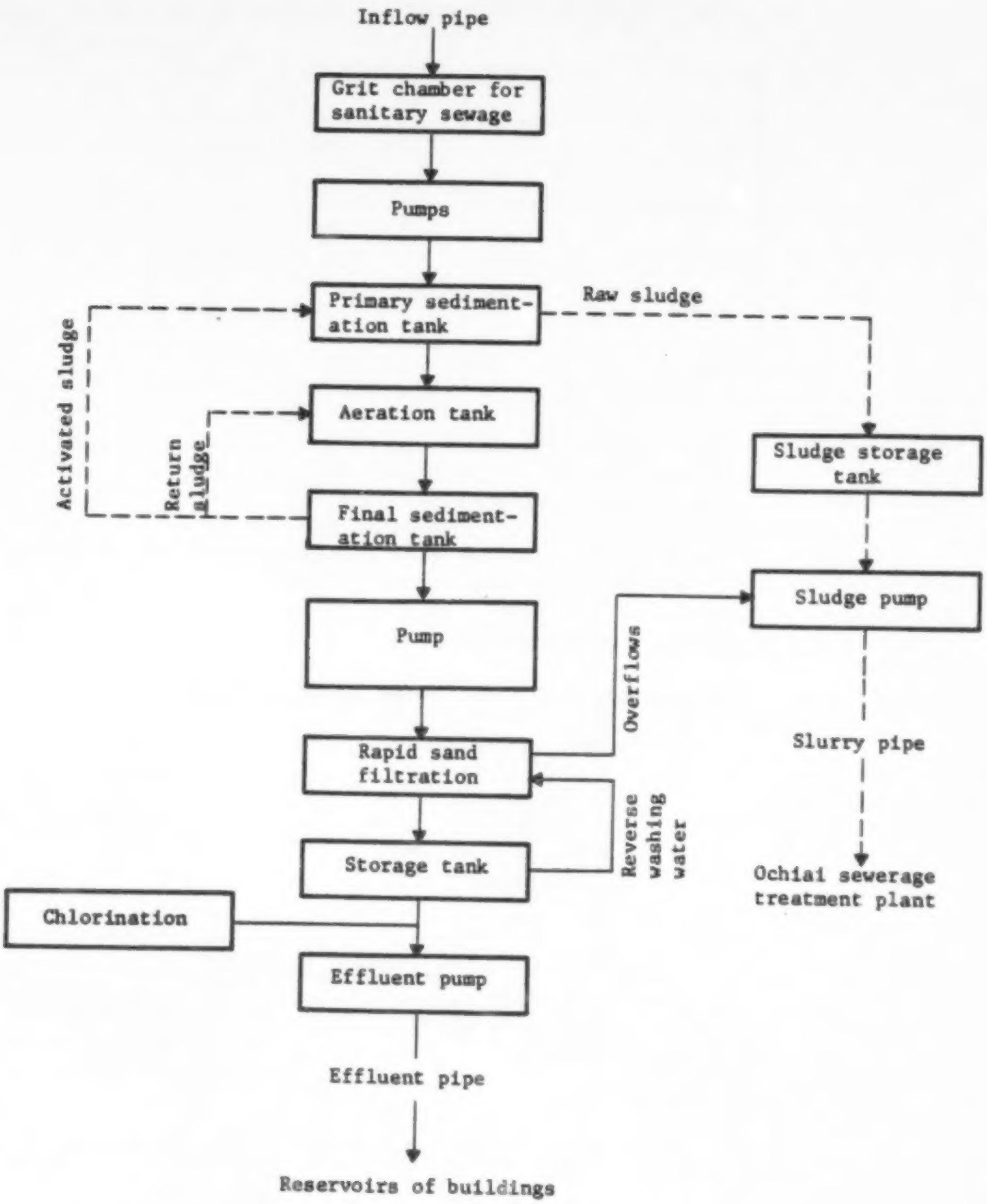
For treatment process, in future, will be proposed "activated sludge process → rapid sand filtration → chrolination". The proposed process is adopted because of it's effectiveness for producing expected water quality and economy. Table-7, 8 shows the effluent quality of Ochiai sewage treatment plant and sand filtrate.

f) Water supply facilities

Flow sheet for first stage is as follows.


Tap water
Reservoir tank
Reuse water
Drinking
Flush toilet
Re-ceive tank
Re-ceive tank
Tap water
Reuse water
P
Pump
Control panel
Storage reservoirs
Water recycling center
Sewage
Sewage
Secondary treatment facility
P
Effluent
Advanced treatment facility
P
Chlorination
P
Sand filtration tank
Pump
Reuse water
Ochiai treatment plant

For second stage, following sewage treatment facilities will plan to be installed in water recycling center.


Inflow pipe
Grit chamber for sanitary sewage
Pumps
Primary sedimentation tank
Raw sludge
Activated sludge
Aeration tank
Return sludge
Sludge storage tank
Final sedimentation tank
Pump
Sludge pump
Overflows
Rapid sand filtration
Slurry pipe
Reverse washing water
Storage tank
Ochiai sewerage treatment plant
Chlorination
Effluent pump
Effluent pipe
Reservoirs of buildings

g) Cost for the project

Total cost for this project was estimated about 6 billion yen. Among them, the cost for the first stage was about 3.2 billion yen.
The source of finance for the construction of the project is national subsidies, general obligation and service charges.
The service charges are collected form the users in proportion to the volume of water consumed.
Water rate is estimated 250 yen per m^3.
Among them, maintenance cost is 117 yen per m^3 and capital cost is 133 yen.

h) Cross connections

To ensure safety, cross connections and misuse must be carefully avoided.
In this project, following procedures were adopted.

① Main pipes and service pipes for reuse of sewage were covered with brown tapes.

② The lids of valves were labelled as reuse of water.

③ Supply pipes in the user's building were also labelled as reclaimed water according to our suggestions.

④ Users were adviced to detect cross connection in their dual supply systems using special dosage.

5. CONCLUSIONS

Reuse of water means that suitable water quality is served for particular use. And it represents the resonable use of energy and resources.

In order to facilitate water reuse, the costs and the space for installation and operation of the treatment plant must be kept to a minimum. Therefore, compactness in the size of equipment and easy maintenance are required.

To cover the shortage of water resources, and the insufficient capacity of sewerage works, and to protect river from pollution, development of new technology for sewage reuse is absolutely essential.

If there is a technical break through in compactness of treatment plant, manpower savings and reduced costs, the concept of waste water reuse may become dominant among the general public.

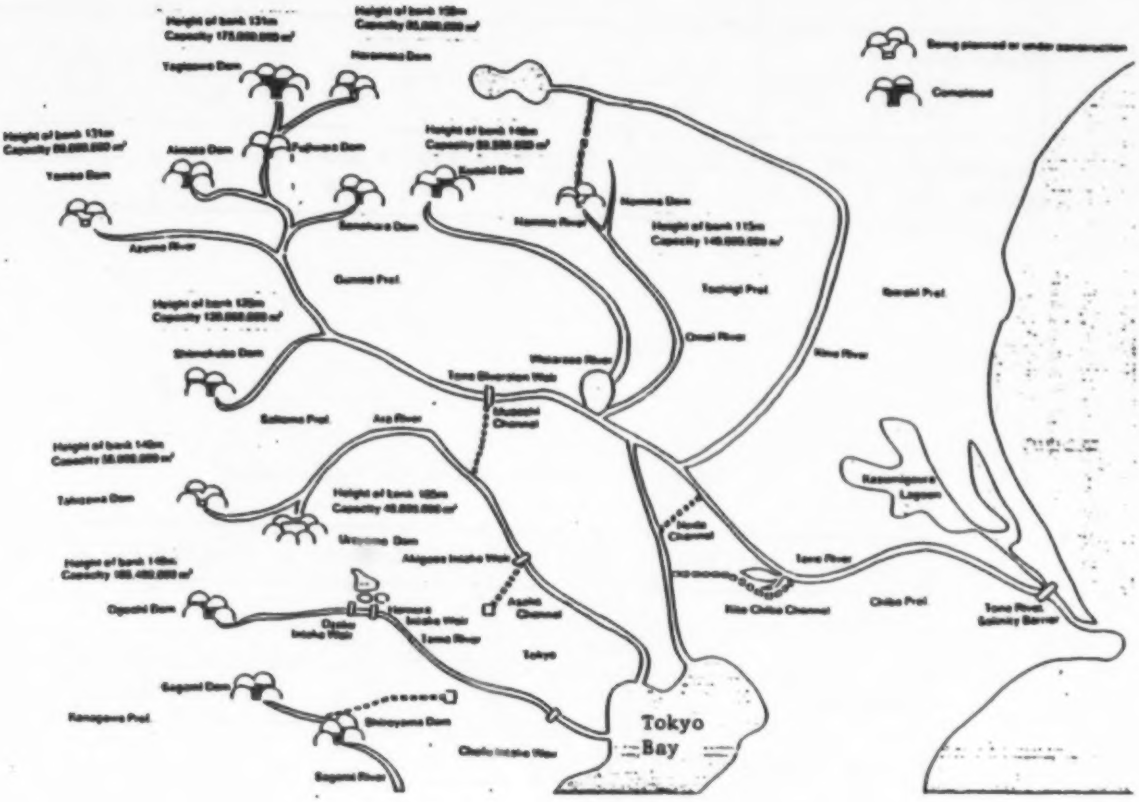

Height of bank 131m
Capacity 175,000,000 m³
Height of bank 158m
Capacity 85,000,000 m³
Naramata Dam
Yagisawa Dam
Being planned or under construction
Completed
Height of bank 131m
Capacity 88,000,000 m³
Aimata Dam
Fujiwara Dam
Yamba Dam
Height of bank 140m
Capacity 50,500,000 m³
Kusaki Dam
Sonohara Dam
Agatsuma River
Gunma Pref
Height of bank 115m
Capacity 140,000,000 m³
Tochigi Pref
Ibaraki Pref
Height of bank 129m
Capacity 130,000,000 m³
Shimokubo Dam
Omoi River
Kinu River
Watarase River
Tone Diversion Weir
Musashi Channel
Saitama Pref
Ara River
Height of bank 140m
Capacity 58,000,000 m³
Takizawa Dam
Height of bank 155m
Capacity 45,000,000 m³
Urayama Dam
Kasumigaura Lagoon
Noda Channel
Akigase Intake Weir
Tone River
Height of bank 149m
Capacity 185,400,000 m³
Asaka Channel
Kita Chiba Channel
Chiba Pref
Tone River Salinity Barrier
Ogochi Dam
Hamura Intake Weir
Ozaku Intake Weir
Tama River
Tokyo
Sagami Dam
Kanagawa Pref
Shiroyama Dam
Tokyo Bay
Chofu Intake Weir
Sagami River

Figure 1. Water Sources for Tokyo

Table 1. Proportion of Usage

	Cooling	Washing	Product treatment	Materials	Temperature regulation	Boilers	Others	Total
Koto area (m^3) (%)	37,005 51.4	25,027 34.8	4,163 5.8	1,220 1.7	395 0.5	64 0.1	4,118 5.7	71,992 100
Johoku(m^3) area (%)	54,763 41.5	33,875 25.7	18,070 13.7	4,955 3.8	3,365 2.6	6,960 5.3	9,717 7.4	131,705 100

Table 2. Use of Water in Buildings

Type of building	Scale	Flushing (%)	Cooling (%)
Office	Big Small	0.3 ∿ 0.5 0.5 ∿ 0.8	
Hospital		0.2 ∿ 0.3	0.3 ∿ 0.4
Department store		0.15 ∿ 0.35	0.3 ∿ 0.5
University	Culture Science	0.35 ∿ 0.7 0.1 ∿ 0.2	

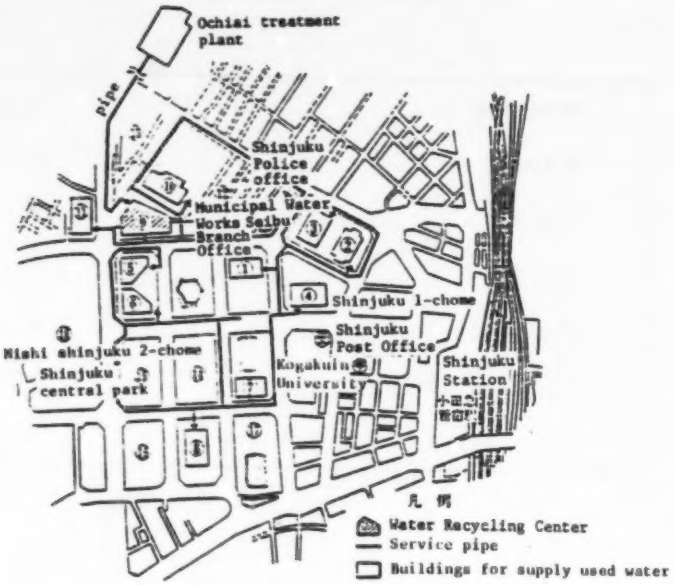
Table 2. Domestic Consumption

(unit: %)

Drinking	20 ∿ 30
Bathing	15 ∿ 20
Washing	5 ∿ 15
Laundry	25
Cleaning	4 ∿ 8
Flushing toilet	20
Car washing and sprinkling	2 ∿ 5

Table 3. Recycling project undergoing

Items / Classification	Number of project	Total water consumption	Recycling quantity	Recycling rate	Reference
In operation	47	42,279	9,931	23.5	2 district wide recycling projects (7 buildings)
Under construction	20	14,975	2,926	19.5	1 district wide recycling project (2 buildings)
Planning	20	22,842	6,857	30.0	3 district wide recycling projects (15 buildings)
Total	87	80,096	19,714	24.6	6 district wide recycling projects (24 buildings)


Ochiai treatment plant
pipe
Shinjuku Police office
Municipal Water Works Seibu Branch Office
Shinjuku 1-chome
Nishi shinjuku 2-chome
Shinjuku central park
Shinjuku Post Office
Kogakuin University
Shinjuku Station
凡 例
Water Recycling Center
Service pipe
Buildings for supply used water

① Shinjuku Mitsui Building

② Yasuda Marine and Fire Insurance Building

③ Shinjuku Nomura Building

④ Shinjuku Center Building

⑤ Shinjuku Daiichi Life Insurance Building

⑥ Odakyu Century Building

⑦ Keio Plaza, Hotel

⑧ Shinjuku NS Building

⑨ Shinjuku International Building

(a) Constructed

⑩ Tokyo Dental University

⑪ Nishi Shinjuku Jofuji District

⑫ Nishi Shinjuku 6-chome East District

⑬ Nishi Shinjuku 6-chome South District

⑭
⑮
⑯
⑰
} Municipal sits

⑱ Shinjuku Central Park

(b) Planning

Figure 2. Shinjuku Sewage Recycling Project Area

Table 4. Water Consumption by Unit Floor Area

(m^3/m^2/d)

Classification	Number of samples	Average (ℓ/m^2/d)	Maximum (ℓ/m^2/d)	Minimum (ℓ/m^2/d)	Standard deviation (ℓ/m^2/d)	Variance
Office	50 51	78.1 10.9	189.2 27.8	19.2 3.1	43.9 5.6	0.562 0.510
School	42 42	19.1 16.1	76.8 113.6	1.2 4.8	17.6 17.4	0.919 1.081
Hospital	22 22	49.8 27.5	78.6 58.5	11.8 13.1	18.2 12.4	0.365 0.451
Department	16 16	161.2 26.0	507.6 85.9	37.4 8.5	121.8 22.4	0.756 0.863
Hotel	6 6	153.5 30.8	350.5 52.1	29.3 12.3	112.6 14.8	0.733 0.481
Factory	32 32	51.5 31.1	493.3 131.3	1.2 2.3	96.8 29.2	1.880 0.934
Miscellaneous	37 37	94.2 24.4	482.2 73.7	1.6 1.7	120.0 18.7	1.274 0.766

(Upper: per unit sits
Lower: per unit floor area)

Table 5. Water Consumption for Usage

(unit: m^3/d)

Items	Shinjuku NS Building	Keio Plaza Main Building	Keio Plaza Southern Building	SS-7	Shinjuku Center Building	Yasuda Marine and Fire Insurance Company	Total
Flushing toilet	641	440	220	719	770	452	3,242
Cooling	0	60	30	0	0	0	90
Car washing	39	20	20	24	5	4	112
Sprinkling	25	0	0	0	0	6	31
Cleansing	19	0	0	0	0	0	19
Washing of hands and face	178	90	45	124	201	177	815
Steam	515	830	405	1,253	551	333	3,787
Bathing	65	430	225	533	0	0	1,253
Store	0	50	55	0	0	0	105
Pool	0	0	0	74	0	0	74
Mis-cellaneous	0	80	40	0	15	0	135
Total	1,482	2,000	1,040	2,727	1,542	872	9,663

Table 6. Water Quality Standard of Industrial Water

Items		Koto area	Johoku area
Water quality	Temperature	less than 27°C	
	Turbidity	less than 15 degree	
	Hydrogen ion	pH 5.8.8.6	
	Chlorine ion	less than 1,500 ppm	less than 200 ppm
	Iron ion	less than 0.7 ppm	less than 0.3 ppm
Pressure		More than 0.5kg/cm^3	

Table 7. Daily Average Water Quality at Ochiai Treatment Plant

(1982)

Samples	Inflow sewage	Primary sedimented effluent	Final sedimented effluent
Temperature	18.5		
Transparency	6	7	37
PH	7.1 ∿ 7.4	7.1 ∿ 7.4	7.0 ∿ 7.4
Vaporized residual	463	391	281
Furnace residual	233	208	198
Furnace reduction	238	182	84
D.S.	302	301	270
S.S.	161	88	12
BOD	148	110	14
COD (alkaline)	55	46	14
COD (acid)	80	63	17
Total-N	28.0	27.0	21.6
NH_3-N	12.2	12.8	11.9
Organic-N	14.6	13.7	7.0
Chlorine	51	52	52
Content of normal hexane extract	18.7		1.0
MBAS	7.3		0.7

Table 8. Water Quality of Sand Filtrate

Items	Value
Temperature (°C)	17.8
Transparency	75
Turbudity	10
Color	15
Odour	none
PH	7.3
SS (mg/ℓ)	5
BOD (mg/ℓ)	7
COD (mg/ℓ)	15
Chlorine (mg/ℓ)	52
MBAS (mg/ℓ)	0.2
Alkalinity (mg/ℓ)	138
Conductivity(μs/h)	595

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